Second Edition

Handbook of Structural Steel Connection Design and Details

AKBAR R. TAMBOLI

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Fasteners and Welds for Structural Connections

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(*Courtesy of The Steel Institute of New York.*)

1.1 Introduction

There are two common ways to connect structural steel members—using bolts or welds. Rivets, while still available, are not currently used for new structures and will not be considered here. This chapter will present the basic properties and requirements for bolts and welds.

Connections are an intimate part of a steel structure and their proper treatment is essential for a safe and economic structure. An intuitive knowledge of how a system will transmit loads (the art of load paths), and an understanding of structural mechanics (the science of equilibrium and limit states), are necessary to achieve connections which are both safe and economic. Chapter 2 will develop this material. This chapter is based on the bolting and welding requirement specifications of the American Institute of Steel Construction (AISC), "Specification for Structural Steel Buildings," 2005, and the American Welding Society Structural Welding Code, D1.1 (2006).

1.2 Bolted Connections

1.2.1 Types of bolts

There are three kinds of bolts used in steel construction. These are high-strength structural bolts manufactured under the American Society for Testing and Materials (ASTM) Specifications A325 and A490, Fig. 1.1,

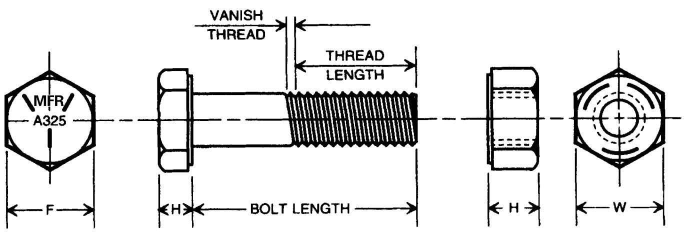


Figure 1.1 High-strength structural-steel bolt and nut.

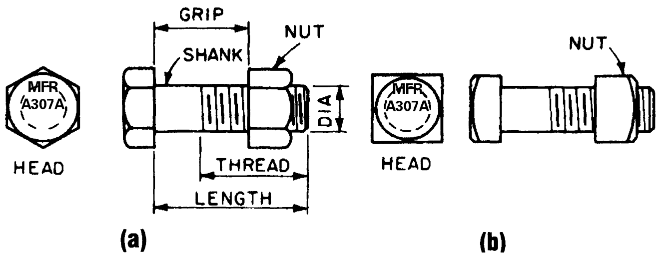


Figure 1.2 Unfinished (machine) or common bolts.

and common bolts manufactured under ASTM A307, Fig. 1.2. The A325 and A490 bolts are *structural bolts* and can be used for any building application. Twist–off-type tension control fasteners manufactured under ASTM F1852 and F2280 are also available and can be treated as subsets of A325 and A490, respectively. A307 bolts, which were referred to previously as *common bolts,* are also variously called *machine bolts, ordinary bolts,* and *unfinished bolts.* The use of these bolts is limited primarily to shear connections in nonfatigue applications.

Structural bolts (A325 and A490) can be installed pretensioned or snug tight. Pretensioned means that the bolt is tightened until a tension force approximately equal to 70 percent of its minimum tensile strength is produced in the bolt. Snug tight is the condition that exists when all plies are in contact. It can be attained by a few impacts of an impact wrench or the full effort of a man using an ordinary spud wrench. Common bolts (A307) can be installed only to the snug-tight condition. There is no recognized procedure for tightening these bolts beyond this point.

Pretensioned structural bolts must be used in certain locations. Section J1.10 of the AISC specification requires that they be used for the following joints:

1. Column splices in all multistory structures more than 125 ft (38 m) in height
2. Connections of all beams and girders to columns and any other beams and girders on which the bracing of columns is dependent in structures more than 125 ft (38 m) in height

3. In all structures carrying cranes of more than 5-ton (50 kN) capacity: roof truss splices and connections of trusses to columns, column splices, column bracing, knee braces, and crane supports
4. Connections for the support of machinery and other live loads that produce impact or reversal of load

Also, AISC Specification section J3.1 requires that A490 bolts subject to tension loads be pretensioned. In all other cases, A307 bolts and snug-tight A325 and A490 bolts can be used.

In general, the use of high-strength structural bolts shall conform to the requirements of the Research Council on Structural Connections (RCSC) “Specification for Structural Joints Using ASTM A325 and A490 Bolts,” 2004. This document, which is specific to A325 and A490 bolts, contains all of the information on design, installation, inspection, washer use, compatible nuts, etc. for these bolts. There is no comparable document for A307 bolts. The RCSC “bolt spec.” was developed in the 1950s to allow the replacement of rivets with bolts.

Many sizes of high-strength bolts are available, as shown in Table 1.1. In general, a connection with a few large-diameter fasteners costs less than one of the same capacity with many small-diameter fasteners. The fewer the fasteners the fewer the number of holes to be formed and the less installation work. Larger-diameter fasteners are generally favorable in connections, because the load capacity of a fastener varies with the square of the fastener diameter. For practical reasons, however, ¾- and ⅞-in-diameter fasteners are usually preferred. Shop and erection equipment is generally set up for these sizes, and workers are familiar with them. It is also advisable to limit the diameter of bolts that must be pretensioned to 1⅛ in since this is the largest diameter tension control (TC) bolt available.

1.2.2 Washer requirements

Washers are generally not required in snug-tightened joints. However, a beveled ASTM F436 washer should be used where the outer face of the

TABLE 1.1 Thread Lengths for High-Strength Bolts

Bolt diameter, in	Nominal thread, in	Vanish thread, in	Total thread, in
½	1.00	0.19	1.19
⅝	1.25	0.22	1.47
¾	1.38	0.25	1.63
⅞	1.50	0.28	1.78
1	1.75	0.31	2.06
1⅛	2.00	0.34	2.34
1¼	2.00	0.38	2.38
1⅜	2.25	0.44	2.69
1½	2.25	0.44	2.69

TABLE 1.2 Washer Requirements for High Strength Bolts

Washer Requirements for Pretensioned or Slip-Critical Joints*

<table>
<tr><th rowspan="2">Bolt type</th><th rowspan="2">Bolt dia. (in)</th><th rowspan="2">Fy< 40</th><th colspan="3">Installation method</th><th colspan="2">Hole in outer ply</th></tr>
<tr><th>Calibrated wrench</th><th>Twist-off tension control</th><th>Direct tension indicator</th><th>OVS or SSL</th><th>LSL</th></tr>
<tr><td>A325</td><td>≤1½</td><td>Not REQ'D</td><td rowspan="3">REQ'D Under turned element</td><td rowspan="3">REQ'D Under nut</td><td rowspan="3">REQ'D See RCSC spec. for location</td><td>REQ'D</td><td>5/16" Plt. washer or Cont. bar</td></tr>
<tr><td rowspan="2">A490</td><td>≤1</td><td rowspan="2">REQ'D†</td><td>REQ'D</td><td rowspan="2">REQ'D w/ 3/8" Plt. washer or Cont. bar</td></tr>
<tr><td>>1</td><td>REQ'D 5/16" thick‡</td></tr>
</table>

*REQ'D indicates a washer conforming to ASTM F436 is required.
†Not required for F2280 with a circular head.
‡A 3/8 in plate washer and an ordinary thickness F436 washer may be used. The plate washer need not be hardened.

bolted parts has a greater slope than 1:20 with respect to a plane normal to the bolt axis. Additionally, an ASTM F436 washer must be provided to cover the hole when a slotted hole occurs in an outer ply. Alternatively a 5/16 in common plate washer can be used to cover the hole.

Washers conforming to ASTM F436 are required in pretensioned and slip-critical joints as indicated in Table 1.2.

1.2.3 Pretensioned and snug-tight bolts

As pointed out in a previous section, pretensioned bolts must be used for certain connections. For other locations, snug-tight bolts should be used because they are cheaper with no reduction in strength. The vast majority of shear connections in buildings can be snug tight, and shear connections are the predominate connection in every building. Also, if common bolts provide the required strength, they should be used because they are less expensive than structural bolts. There is no danger of interchanging the two types because all bolts are required to have clear identifying marks, see Fig. 1.1 for structural bolts and Fig. 1.2 for common bolts.

1.2.4 Bearing-type versus slip-critical joints

Connections made with high-strength bolts may be slip-critical (material joined being clamped together by the tension induced in the bolts by tightening them) or bearing-type (material joined being restricted from moving primarily by the bolt shank). In bearing-type connections, bolt threads may be included in or excluded from the shear plane. Different design strengths are used for each condition. Also, bearing-type connections may be either pretensioned or snug-tight, subject to the limitations already discussed. Snug-tight bolts are much more economical to install and should be used where permitted. The slip-critical connection is the most expensive, because it requires that the faying surfaces be free of paint, grease, and oil, or that a special paint be used. Hence this type of connection should be used only where required by the governing design specification, for example, where it is undesirable to have the bolts slip into bearing or where stress reversal could cause slippage. The 2005 AISC specification requires the use of slip-critical connections when

(a) Bolts are installed in oversized holes

(b) Bolts are installed in slotted holes with the direction of the load parallel to the slot

The RCSC specification further requires slip-critical connections for

(c) Joints that are subject to fatigue load with reversal of the loading direction

(d) Joints in which slip at the faying surfaces would be detrimental to the performance of the structure.

The 2005 AISC specification includes provisions for designing slip-critical connections at either the strength level or the serviceability level. As the name implies the serviceability limit state assumes that slip in the joint would affect only the serviceability of the structure and not lead to collapse. The minimal slip that could occur in a joint with standard holes is generally thought to be negligible. Therefore the specification recommends that joints utilizing standard holes or slots perpendicular to the load should be designed at the serviceability level.

In contrast, connections where slip at the joint could lead to a collapse, should be designed considering slip as a strength level limit state. The specification conservatively recommends designing joints utilizing oversized holes or slots parallel to the direction of the load at the strength level. However, the choice of strength versus serviceability is ultimately left to the discretion of engineer. If for example during the design of the main members the P-Δ effects resulting from joint slip are considered, the connection could safely be designed with slip as a serviceability limit state.

Threads included in shear planes. The bearing-type connection with threads in shear planes is most frequently used. Since location of threads is not restricted, bolts can be inserted from either side of a connection.

Either the head or the nut can be the element turned. Paint of any type is permitted on the faying surfaces.

Threads excluded from shear planes. The bearing-type connection with threads excluded from shear planes is the most economical high-strength bolted connection, because fewer bolts generally are needed for a given required strength. There can be difficulties involved in excluding the threads from the shear planes when either one or both of the outer plies of the joint is thin. The location of the thread runout or vanish depends on which side of the connection the bolt is entered and whether a washer is placed under the head or the nut. This location is difficult to control in the shop but even more so in the field. However, since for a given diameter of bolt the thread length is constant, threads can often be excluded in heavy joints with no additional effort.

Total nominal thread lengths and vanish thread lengths for high-strength bolts are given in Table 1.1. It is common practice to allow the last ⅛ in of vanish thread to extend across a single shear plane.

In order to determine the required bolt length, the value shown in Table 1.3 should be added to the grip (that is, the total thickness of all connected material, exclusive of washers). For each hardened flat washer that is used, add 5⁄32 in and for each beveled washer, add 5⁄16 in. The tabulated values provide appropriate allowances for manufacturing tolerances and also provide for full thread engagement with an installed heavy hex nut. The length determined by the use of Table 1.3 should be adjusted to the next longer ¼ in length.

1.2.5 Bolts in combination with welds

Due to differences in the rigidity and ductility of bolts as compared to welds, sharing of loads between bolts and welds should generally be avoided. However, the specification does not completely prohibit it.

In new construction, 50 percent of the bearing-type strength of bolts can be assumed to be effective when sharing load with longitudinally

TABLE 1.3 Lengths to Be Added to Grip

Nominal bolt size, in	Addition to grip for determination of bolt length, in
½	11⁄16
⅝	⅞
¾	1
⅞	1⅛
1	1¼
1⅛	1½
1¼	1⅝
1⅜	1¾
1½	1⅞

loaded welds. Longitudinal loading is specified since welds become significantly less ductile as the loading moves from longitudinal to transverse. This provision assumes that the direction of load is known, which may not be the case if an eccentric load is present.

In welded alterations to structures, existing rivets and high-strength bolts tightened to the requirements for slip-critical connections are permitted for carrying stresses resulting from loads present at the time of alteration. The welding needs to be adequate only to carry the additional stress.

1.2.6 Standard, oversized, short-slotted, and long-slotted holes

The AISC specification requires that standard holes for bolts be 1⁄16 in larger than the nominal fastener diameter. In computing net area or a tension member, the diameter of the hole should be taken 1⁄16 in larger than the hole diameter.

Holes can be punched, drilled, or thermally cut. Punching usually is the most economical method. To prevent excessive damage to material around the hole, however, the specifications limit the maximum thickness of material in which holes may be punched full size. These limits are summarized in Table 1.4.

In buildings, holes for thicker material may be either drilled from the solid or subpunched and reamed. The die for all subpunched holes and the drill for all subdrilled holes should be at least 1⁄16 in smaller than the nominal fastener diameter.

Oversize holes can be used in slip-critical connections, and the oversize hole can be in some or all the plies connected. The oversize holes are 3⁄16 in larger than the bolt diameter for bolts 5⁄8 to 7⁄8 in in diameter. For bolts 1 in in diameter, the oversize hole is 1⁄4 in larger and for bolts 1 1⁄8 in in diameter and greater, the oversize hole will be 5⁄16 in larger.

Short-slotted holes can be used in any or all the connected plies. The load has to be applied 80 to 100° normal to the axis of the slot in bearing-type connections. Short slots can be used without regard to the direction

TABLE 1.4 Maximum Material Thickness (in) for Punching Fastener Holes*

Type of steel	AISC
A36 steel	$d + 1/8$†
High-strength steels	$d + 1/8$†
Quenched and tempered steels	1⁄2‡

*Unless subpunching or subdrilling and reaming are used.

†$d \times$ fastener diameter, in.

‡A514 steel.

of the applied load when slip-critical connections are used. The short slots for ⅝- to ⅞-in-diameter bolts are 1⁄16 in larger in width and 1⁄14 in larger in length than the bolt diameter. For bolts 1 in in diameter, the width is 1⁄16 in larger and the length 5⁄16 in larger and for bolts 1⅛ in diameter and larger, the slot will be 1⁄16 in larger in width and ⅜ in longer in length.

Long slots have the same requirement as the short-slotted holes, except that the long slot has to be in only one of the connected parts at the faying surface of the connection. The width of all long slots for bolts is 1⁄16 in greater than the bolt diameter, and the length of the long slots for ⅝-in-diameter bolts is 5⁄16 in greater, for ¾ -in-diameter bolts 1⅛ in greater, for ⅞-in-diameter bolts 15⁄16 in greater, for 1-in-diameter bolts 1½ in greater, and for 1⅛-in-diameter and larger bolts, 2½ times diameter of bolt.

When finger shims are fully inserted between the faying surfaces of load transmitting parts of the connections, this is not considered as a long-slot connection.

1.2.7 Edge distances and spacing of bolts

Minimum distances from centers of fasteners to any edges are given in Table 1.5.

The AISC specification has provisions for minimum edge distance: The distance from the center of a standard hole to an edge of a connected part should not be less than the applicable value from Table 1.5.

Maximum edge distances are set for sealing and stitch purposes. The AISC specification limits the distance from center of fastener to nearest

TABLE 1.5 Minimum Edge Distances* (in) for Fastener Holes in Steel for Buildings

Fastener diameter, in	At sheared edges	At rolled edges of plates, shapes, or bars or gas-cut edges†
½	⅞	¾
⅝	1⅛	⅞
¾	1¼	1
⅞	1½‡	1⅛
1	1¾‡	1¼
1⅛	2	1½
1¼	2¼	1⅝
Over 1¼	1¾ d§	1¼d§

*Lesser distances are permitted if bolt edge tear-out is checked (J3.10).

†All edge distances in this column may be reduced ⅛ in when the hole is at point where stress does not exceed 25 percent of the maximum allowed stress in the element.

‡These may be 1¼ in. at the ends of beam connection angles.

§d = fastener diameter in.

SOURCE: From AISC "Specification for Structural Steel Buildings."

edge of parts in contact to 12 times the thickness of the connected part, with a maximum of 6 in. For unpainted weathering steel, the maximum is 7 in or 14 times the thickness of the thinner plate. For painted or unpainted members not subject to corrosion, the maximum spacing is 12 in or 24 times the thickness of the thinner plate.

Pitch is the distance (in) along the line of principal stress between centers of adjacent fasteners. It may be measured along one or more lines of fasteners. For example, suppose bolts are staggered along two parallel lines. The pitch may be given as the distance between successive bolts in each line separately. Or it may be given as the distance, measured parallel to the fastener lines, between a bolt in one line and the nearest bolt in the other line.

Gage is the distance (in) between adjacent lines of fasteners along which pitch is measured or the distance (in) from the back of an angle or other shape to the first line of fasteners.

The minimum distance between centers of fasteners should usually be at least 3 times the fastener diameter. However, the AISC specification permits a minimum spacing of 2⅔ times the fastener diameter.

Limitations also are set on maximum spacing of fasteners, for several reasons. In built-up members, stitch fasteners, with restricted spacings, are used between components to ensure uniform action. Also, in compression members such fasteners are required to prevent local buckling.

Designs should provide ample clearance for tightening high-strength bolts. Detailers who prepare shop drawings for fabricators generally are aware of the necessity for this and can, with careful detailing, secure the necessary space. In tight situations, the solution may be staggering of holes (Fig. 1.3), variations from standard gages (Fig. 1.4), use of knife-type connections, or use of a combination of shop welds and field bolts.

Minimum clearances for tightening high-strength bolts are indicated in Fig. 1.5 and Table 1.6.

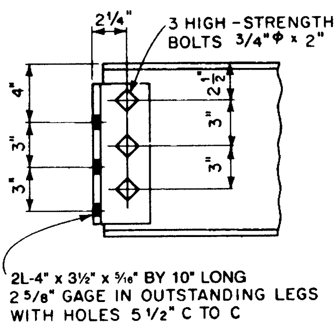


Figure 1.3 Staggered holes provide clearance for high-strength bolts.

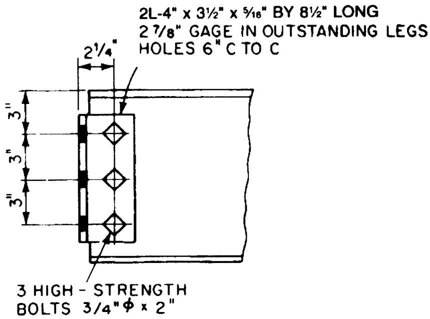


Figure 1.4 Increasing the gage in framing angles.

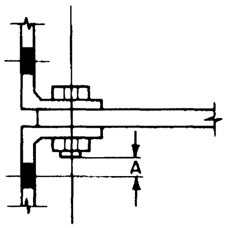


Figure 1.5 The usual minimum clearances.

TABLE 1.6 Clearances for High-Strength Bolts

Bolt diameter, in	Nut height, in	Usual minimum clearance, *A*, in	Minimum clearance for twist-off bolts, *A*, in	
			Small tool	Large tool
⅝	39⁄64	1	1⅝	—
¾	47⁄64	1¼	1⅝	1⅞
⅞	55⁄64	1⅜	1⅝	1⅞
1	63⁄64	1 7⁄16		1⅞
1⅛	1 7⁄64	1 9⁄16		—
1¼	1 7⁄32	1 11⁄16		—

1.2.8 Installation

All parts of a connection should be held tightly together during installation of fasteners. Drifting done during assembling to align holes should not distort the metal or enlarge the holes. Holes that must be enlarged to admit fasteners should be reamed. Poor matching of holes is cause for rejection.

For connections with high-strength bolts, surfaces, when assembled, including those adjacent to bolt heads, nuts, and washers, should be free of scale, except tight mill scale. The surfaces also should be free of defects that would prevent solid seating of the parts, especially dirt, burrs, and other foreign material. Contact surfaces within slip-critical joints should be free of oil, paint (except for qualified paints), lacquer, and rust inhibitor.

High-strength bolts usually are tightened with an impact or TC wrench. Only where clearance does not permit its use will bolts be hand-tightened

Each high-strength bolt should be tightened so that when all fasteners in the connection are tight it will have the total tension (kips) for its diameter. Tightening should be done by one of the following methods, as given in the RCSC specifications (2004).

Calibrated-wrench method. When a calibrated wrench is used, it must be set to cut off tightening when the required tension has been exceeded by 5 percent. The wrench should be tested periodically (at least daily on a minimum of three bolts of each diameter being used). For this purpose, a calibrating device that gives the bolt tension directly should be used. In particular, the wrench should be calibrated when bolt size or length of air hose is changed. When bolts are tightened, bolts previously tensioned may become loose because of compression of the connected parts. The calibrated wrench should be reapplied to bolts previously tightened to ensure that all bolts are tensioned to the prescribed values.

Turn-of-the-nut method. When the turn-of-the-nut method is used, tightening may be done by impact or hand wrench. This method involves the following three steps:

1. *Fit up of connection*. Enough bolts are tightened a sufficient amount to bring contact surfaces together. This can be done with fit-up bolts, but it is more economical to use some of the final high-strength bolts.
2. *Snug tightening of bolts*. All high-strength bolts are inserted and made snug-tight (tightness obtained with a few impacts of an impact wrench or the full effort of a person using an ordinary spud wrench). While the definition of snug-tight is rather indefinite, the condition can be observed or learned with a tension-testing device.
3. *Nut rotation from snug-tight position*. All bolts are tightened by the amount of nut rotation specified in Table 1.7. If required by bolt-entering and wrench-operation clearances, tightening, including by the calibrated-wrench method, may be done by turning the bolt while the nut is prevented from rotating.

Direct tension indicator. The direct tension indicator (DTI) hardened-steel load-indicator washer has dimples on the surface of one face of the washer. When the bolt is tensioned, the dimples depress to the

TABLE 1.7 Number of Nut or Bolt Turns from Snug-Tight Condition for High-Strength Bolts*

Bolt length (Fig. 1.1)	Both faces normal to bolt axis	Slope of outer faces of bolted parts: One face normal to bolt axis and the other sloped†	Slope of outer faces of bolted parts: Bolt faces sloped†
Up to 4 diameters	⅓	½	⅔
Over 4 diameters but not more than 8 diameters	½	⅔	⅚
Over 5 diameters but not more than 12 diameters‡	⅔	⅚	1

*Nut rotation is relative to the bolt regardless of whether the nut or bolt is turned. For bolts installed by ½ turn and less, the tolerance should be ±30°. For bolts installed by ⅔ turn and more, the tolerance should be ±45°. This table is applicable only to connections in which all material within the grip of the bolt is steel.

†Slope is not more than 1:20 from the normal to the bolt axis, and a beveled washer is not used.

‡No research has been performed by RCSC to establish the turn-of-the-nut procedure for bolt lengths exceeding 12 diameters. Therefore, the required rotation should be determined by actual test in a suitable tension-measuring device that simulates conditions of solidly fitted steel.

manufacturer's specification requirements, and proper pretension can be verified by the use of a feeler gage. Special attention should be given to proper installation of flat hardened washers when load-indicating washers are used with bolts installed in oversize or slotted holes and when the load-indicating washers are used under the turned element.

Twist-off-type tension-control bolts. The twist off or TC bolt is a bolt with an extension to the actual length of the bolt. This extension will twist off when torqued to the required tension by a special torque gun. The use of TC bolts have increased for both shop and fieldwork, since they allow bolts to be tightened from one side, without restraining the element on the opposite face. A representative sample of at least three TC assemblies for each diameter and grade of fastener should be tested in a calibration device to demonstrate that the device can be torqued to 5 percent greater tension than that required.

For all pretensioning installation methods bolts should first be installed in all holes and brought to the snug-tight condition. All fasteners should then be tightened, progressing systematically from the most rigid part of the connection to the free edges in a manner that will minimize relaxation of previously tightened fasteners. In some cases, proper tensioning of the bolts may require more than a single cycle of systematic tightening.

An excellent source of information on bolt installation is the *Structural Bolting Handbook* (2006).

1.3 Welded Connections

Welded connections are used because of their simplicity of design, fewer parts, less material, and decrease in shop handling and fabrication operations. Frequently, a combination of shop welding and field bolting is advantageous. With connection angles shop-welded to a beam, field connections can be made with high-strength bolts without the clearance problems that may arise in an all-bolted connection.

Welded connections have a rigidity that can be advantageous if properly accounted for in design. Welded trusses, for example, deflect less than bolted trusses, because the end of a welded member at a joint cannot rotate relative to the other members there. If the end of a beam is welded to a column, the rotation there is practically the same for column and beam.

A disadvantage of welding, however, is that shrinkage of large welds must be considered. It is particularly important in large structures where there will be an accumulative effect.

Properly made, a properly designed weld is stronger than the base metal. Improperly made, even a good-looking weld may be worthless. Properly made, a weld has the required penetration and is not brittle.

Prequalified joints, welding procedures, and procedures for qualifying welders are covered by AWS D1.1, *Structural Welding Code—Steel,* American Welding Society (2006). Common types of welds with structural steels intended for welding when made in accordance with AWS specifications can be specified by note or by symbol with assurance that a good connection will be obtained.

In making a welded design, designers should specify only the amount and size of weld actually required. Generally, a $\frac{5}{16}$-in weld is considered the maximum size for a single pass. A $\frac{3}{8}$-in weld, while only $\frac{1}{16}$-in larger, requires three passes and engenders a great increase in cost.

The cost of fit-up for welding can range from about one-third to several times the cost of welding. In designing welded connections, therefore, designers should consider the work necessary for the fabricator and the erector in fitting members together so they can be welded.

1.3.1 Types of welds

The main types of welds used for structural steel are fillet, groove, plug, and slot. The most commonly used weld is the fillet. For light loads, it is the most economical, because little preparation of material is required. For heavy loads, groove welds are the most efficient, because the full strength of the base metal can be obtained easily. Use of plug and slot welds generally is limited to special conditions where fillet or groove welds are not practical.

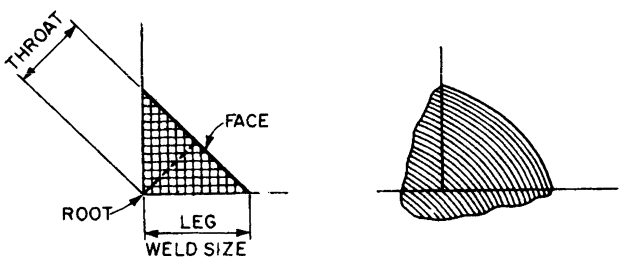
More than one type of weld may be used in a connection. If so, the allowable capacity of the connection is the sum of the effective capacities of each type of weld used, separately computed with respect to the axis of the group.

Tack welds may be used for assembly or shipping. They are not assigned any stress-carrying capacity in the final structure. In some cases, these welds must be removed after final assembly or erection.

Fillet welds have the general shape of an isosceles right triangle (Fig. 1.6). The size of the weld is given by the length of leg. The strength is determined by the throat thickness, the shortest distance from the root (intersection of legs) to the face of the weld. If the two legs are unequal, the nominal size of the weld is given by the shorter of the legs. If welds are concave, the throat is diminished accordingly, and so is the strength.

Fillet welds are used to join two surfaces approximately at right angles to each other. The joints may be lap (Fig. 1.7) or tee or corner (Fig. 1.8). Fillet welds also may be used with groove welds to reinforce corner joints. In a skewed tee joint, the included angle of weld deposit may vary up to 30° from the perpendicular, and one corner of the edge to be connected may be raised, up to $\frac{3}{16}$ in. If the separation is greater than $\frac{1}{16}$ in, the weld leg must be increased by the amount of the root opening. A further discussion of this is continued in Sec. 1.3.7.

Groove welds are made in a groove between the edges of two parts to be joined. These welds generally are used to connect two plates lying in the same plane (butt joint), but they also may be used for tee and corner joints.



(a) (b)

Figure 1.6 Fillet weld: (*a*) theoretical cross section and (*b*) actual cross section.

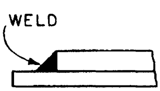
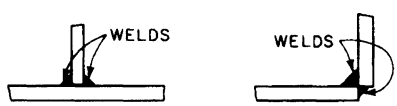


Figure 1.7 Welded lap joint.



(a) (b)

Figure 1.8 (*a*) Tee joint and (*b*) corner joint.

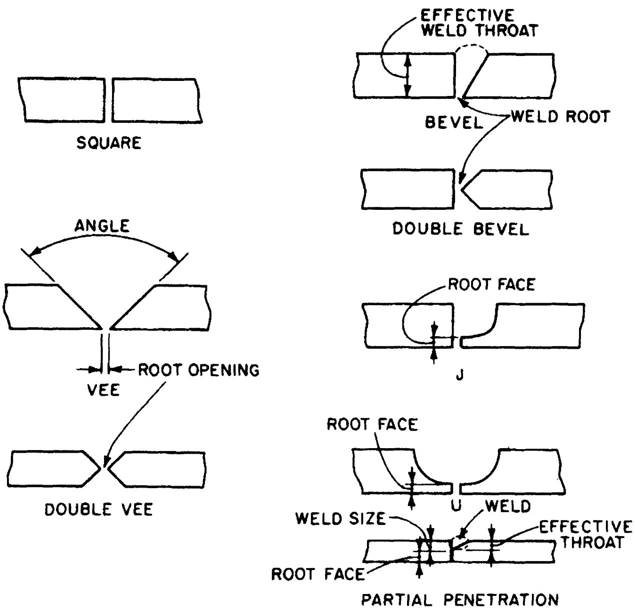


Figure 1.9 Groove welds.

Standard types of groove welds are named in accordance with the shape given the edges to be welded: square, single V, double V, single bevel, double bevel, single U, double U, single J, and double J (Fig. 1.9). Edges may be shaped by flame cutting, arc-air gouging, or edge planing. Material up to ⅜ in thick, however, may be groove-welded with square-cut edges, depending on the welding process used.

Groove welds should extend the full width of the parts joined. Intermittent groove welds and butt joints not fully welded throughout the cross section are prohibited.

Groove welds also are classified as complete-penetration and partial-penetration welds.

In a *complete-joint-penetration weld,* the weld material and the base metal are fused throughout the depth of the joint. This type of weld is made by welding from both sides of the joint or from one side to a backing bar. When the joint is made by welding from both sides, the root of the first-pass weld is chipped or gouged to sound metal before the weld on the opposite side, or back pass, is made. The throat dimension of a complete-joint-penetration groove weld, for stress computations, is the full thickness of the thinner part joined, exclusive of weld reinforcement.

Partial-joint-penetration welds should be used when forces to be transferred are less than those requiring a complete-joint-penetration weld. The edges may not be shaped over the full joint thickness, and the depth of the weld may be less than the joint thickness (Fig. 1.11). But even if the edges are fully shaped, groove welds made from one side without a backing bar or made from both sides without back gouging are considered partial-joint-penetration welds. They are often used for splices in building columns carrying axial loads only.

Plug and slot welds are used to transmit shear in lap joints and to prevent buckling of lapped parts. In buildings, they also may be used to join components of built-up members. (Plug or slot welds, however, are not permitted on A514 steel.) The welds are made, with lapped parts in contact, by depositing weld metal in circular or slotted holes in one part. The openings may be partly or completely filled, depending on their depth. Load capacity of a plug or slot completely welded equals the product of hole area and available design stress. Unless appearance is a main consideration, a fillet weld in holes or slots is preferable.

Economy in selection. In selecting a weld, designers should consider not only the type of joint but also the labor and volume of weld metal required. While the strength of a fillet weld varies with size, the volume of metal varies with the square of the size. For example, a ½-in fillet weld contains 4 times as much metal per inch of length as a ¼-in weld but is only twice as strong. In general, a smaller but longer fillet weld costs less than a larger but shorter weld of the same capacity.

Furthermore, small welds can be deposited in a single pass. Large welds require multiple passes. They take longer, absorb more weld metal, and cost more. As a guide in selecting welds, Table 1.8 lists the

TABLE 1.8 Number of Passes for Welds

		Single-bevel groove welds (backup weld not included)		Single-bevel groove welds (backup weld not included)		
Weld size,* in	Fillet welds	30° bevel	45° bevel	30° open	60° open	90° open
3/16	1					
¼	1	1	1	2	3	3
5/16	1					
⅜	3	2	2	3	4	6
7/16	4					
½	4	2	2	4	5	7
⅝	6	3	3	4	6	8
¾	8	4	5	4	7	9
⅞		5	8	5	10	10
1		5	11	5	13	22
1⅛		7	11	9	15	27
1¼		8	11	12	16	32
1⅜		9	15	13	21	36
1½		9	18	13	25	40
1¾		11	21			

*Plate thickness for groove welds.

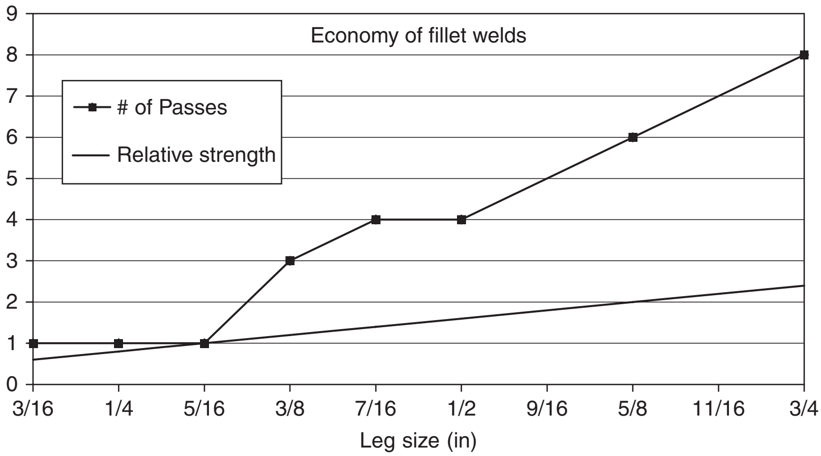


Figure 1.10 Relationship of number of passes to strength.

number of passes required for some frequently used types of welds. This table is only approximate. The actual number of passes can vary depending on the welding process used. Figure 1.10 shows the number of passes and fillet weld strength. It can be seen that cost, which is proportional to the number of passes increases much faster than strength.

Double-V and double-bevel groove welds contain about half as much weld metal as single-V and single-bevel groove welds, respectively (deducting effects of root spacing). Cost of edge preparation and added labor of gouging for the back pass, however, should be considered. Also, for thin material, for which a single weld pass may be sufficient, it is uneconomical to use smaller electrodes to weld from two sides. Furthermore, poor accessibility or less favorable welding position (Sec. 1.3.4) may make an unsymmetrical groove weld more economical, because it can be welded from only one side.

When bevel or V grooves can be flame-cut, they cost less than J and U grooves, which require planning or arc-air gouging.

For a given size of fillet weld, the cooling rate is faster and the restraint is greater with thick plates than with thin plates. To prevent cracking due to resulting internal stresses, the AISC Specification section J2.2 sets minimum sizes for fillet welds depending on plate thickness, see Table 1.9.

To prevent overstressing of base material at a fillet weld the maximum weld size is limited by the strength of the adjacent base metal.

A limitation is also placed on the maximum size of fillet welds along edges. One reason is that edges of rolled shapes are rounded, and weld thickness consequently is less than the nominal thickness of the part.

TABLE 1.9 Minimum Plate Thickness for Fillet Welds

Minimum size of fillet welds,* in	Thickness of thinner part joined, in†	Minimum plate thickness for fillet welds on each side of the plate, in	
		36-ksi steel	50-ksi steel
⅛‡	To ¼ inclusive	0.213	0.190
3/16	Over ¼ to ½	0.320	0.286
¼	Over ½ to ¾	0.427	0.381
5/16	Over ¾	0.534	0.476

*Single pass fillets must be used.
†Plate thickness is the thickness of the thinner part joined.
‡Minimum weld size for structures subjected to dynamic loads is 3/16 in.

Another reason is that if weld size and plate thickness are nearly equal, the plate comer may melt into the weld, reducing the length of weld leg and the throat. Hence the AISC specification requires in section J2.2b the following: *Along edges of material less than ¼ in thick, maximum size of fillet weld may equal material thickness. But along edges of material ¼ in or more thick, the maximum size should be 1/16 in less than the material thickness.*

Weld size may exceed this, however, if drawings definitely show that the weld is to be built out to obtain full throat thickness. AWS D1.1 requires that the minimum-effective length of a fillet weld be at least 4 times the nominal size, or else the weld must be considered not to exceed 25 percent of the effective length.

Subject to the preceding requirements, intermittent fillet welds maybe used in buildings to transfer calculated stress across a joint or faying surfaces when the strength required is less than that developed by a continuous fillet weld of the smallest permitted size. Intermittent fillet welds also may be used to join components of built-up members in buildings.

Intermittent welds are advantageous with light members where excessive welding can result in straightening costs greater than the cost of welding. Intermittent welds often are sufficient and less costly than continuous welds (except girder fillet welds made with automatic welding equipment).

For groove welds, the weld lengths specified on drawings are effective weld lengths. They do not include distances needed for start and stop of welding. These welds must be started or stopped on run-off pads beyond the effective length. The effective length of straight fillet welds is the overall length of the full size fillet. No reduction in effective length need be taken in design calculations to allow for the start or stop weld crater.

To avoid the adverse effects of starting or stopping a fillet weld at a corner, welds extending to corners should be returned continuously around the corners in the same plane for a distance of at least twice the weld size.

This applies to side and top fillet welds connecting brackets, beam seats, and similar connections, on the plane about which bending moments are computed. End returns should be indicated on design and detail drawings.

Filet welds deposited on opposite sides of a common plane of contact between two parts must be interrupted at a corner common to both welds. An exception to this requirement must be made when seal welding parts prior to hot-dipped galvanizing.

If longitudinal fillet welds are used alone in end connections of flat-bar tension members, the length of each fillet weld should at least equal the perpendicular distance between the welds.

In material $\frac{5}{8}$ in or less thick, the thickness of plug or slot welds should be the same as the material thickness. In material greater than $\frac{5}{8}$ in thick, the weld thickness should be at least half the material thickness but not less than $\frac{5}{8}$ in.

The diameter of the hole for a plug weld should be at least equal to the depth of the hole plus $\frac{5}{16}$ in, but the diameter should not exceed $2\frac{1}{4}$ times the thickness of the weld.

Thus, the hole diameter in $\frac{3}{4}$-in plate could be a minimum of $\frac{3}{4} + \frac{5}{16} = 1\frac{1}{16}$ in. The depth of weld metal would be at least $\frac{5}{8}$ in $> (\frac{1}{2} \times \frac{3}{4} = \frac{3}{8}$ in).

Plug welds may not be spaced closer center-to-center than 4 times the hole diameter.

The length of the slot for a slot weld should not exceed 10 times the thickness of the weld. The width of the slot should not be less than the thickness of the part containing it plus $\frac{5}{16}$ in rounded to the next larger $\frac{1}{6}$ in, but the width should not exceed $2\frac{1}{4}$ times the weld thickness.

Thus, the width of the slot in $\frac{3}{4}$-in plate could be a minimum of $\frac{3}{4} + \frac{5}{16} = 1\frac{1}{16}$ in. The weld metal depth would be at least $\frac{5}{8}$ in $> (\frac{1}{2} \times \frac{3}{4} = \frac{3}{8}$ in). The slot could be up to $10 \times \frac{5}{8} = 6\frac{1}{4}$ in long.

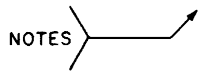
Slot welds may be spaced no closer than 4 times their width in a direction transverse to the slot length. In the longitudinal direction, center-to-center spacing should be at least twice the slot length.

1.3.2 Welding symbols

These should be used on drawings to designate welds and provide pertinent information concerning them. The basic parts of a weld symbol are a horizontal line and an arrow:

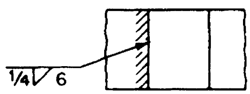
Extending from either end of the line, the arrow should point to the joint in the same manner as the electrode would be held to do the welding.

Welding symbols should clearly convey the intent of the designer. For this purpose, sections or enlarged details may have to be drawn to show the symbols, or notes may be added. Notes may be given as part of welding symbols or separately. When part of a symbol, the note should be placed inside a tail at the opposite end of the line from the arrow:

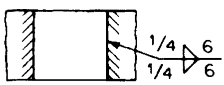


The type and length of weld are indicated above or below the line. If noted below the line, the symbol applies to a weld on the arrow side of the point, the side to which the arrow points. If noted above the line, the symbol indicates that the other side, the side opposite the one to which the arrow points (not the far side of the assembly), is to be welded.

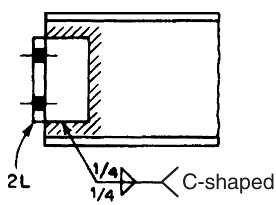
A fillet weld is represented by a right triangle extending above or below the line to indicate the side on which the weld is to be made. The vertical leg of the triangle is always on the left.



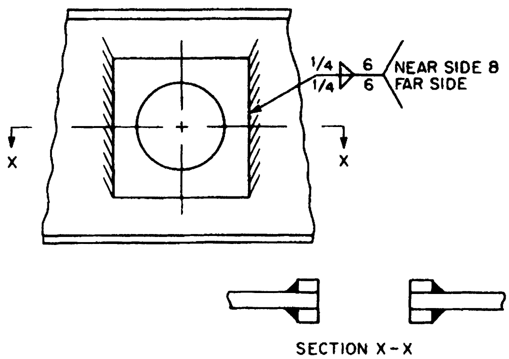
The preceding symbol indicates that a ¼-in fillet weld 6 in long is to be made on the arrow side of the assembly. The following symbol requires a ¼-in fillet weld 6 in long on both sides:



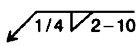
If a weld is required on the far side of an assembly, it may be assumed necessary from symmetry, shown in sections or details, or explained by a note in the tail of the welding symbol. For connection angles at the end of a beam, far-side welds generally are assumed:



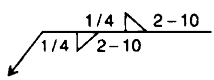
The length of the weld is not shown on the symbol in this case because the connection requires a continuous weld for the full length of each angle on both sides of the angle. Care must be taken not to omit the length unless a continuous full-length weld is wanted. "Continuous" should be written on the weld symbol to indicate length when such a weld is required. In general, a tail note is advisable to specify welds on the far side, even when the welds are the same size.



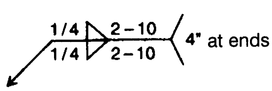
For many members, a stitch or intermittent weld is sufficient. It may be shown as



This symbol calls for ¼-in fillet welds on the arrow side. Each weld is to be 2 in long. Spacing of welds is to be 10 in center-to-center. If the welds are to be staggered on the arrow and other sides, they can be shown as

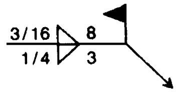


Usually, intermittent welds are started and finished with a weld at least twice as long as the length of the stitch welds. This information is given in a tail note:



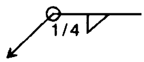
In the previous three figures, intermittent fillets are shown as, for example 2-10. This is the notation recommended by AWS, but it can lead to confusion on shop drawings, where dimensions are given in feet and inches as for instance, 2 ft-10, with no inch symbol. Therefore, 2-10 on a weld symbol could be mistaken as 2 ft, 10 in rather than 2 in at 10 in. It would be less ambiguous to use the "at" symbol, @, rather than the hyphen, -. Then the weld symbol would read 2 @ 10, which is unambiguous.

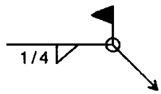
When the welding is to be done in the field rather than in the shop, a triangular flag should be placed at the intersection of arrow and line:



This is important in ensuring that the weld will be made as required. Often, a tail note is advisable for specifying field welds.

A continuous weld all around a joint is indicated by a small circle around the intersection of line and arrow:

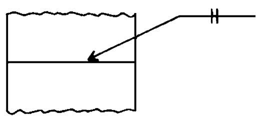




Such a symbol would be used, for example, to specify a weld joining a pipe column to a base plate. The all-around symbol, however, should not be used as a substitute for computation of the actual weld length required. Note that the type of weld is indicated below the line in the all-around symbol, regardless of shape or extent of joint.

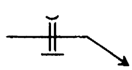
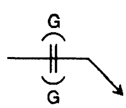
The preceding devices for providing information with fillet welds also apply to groove welds. In addition, groove-weld symbols must designate material preparation required. This often is best shown on a cross section of the joint.

A square-groove weld (made in thin material) without root opening is indicated by



Length is not shown on the welding symbol for groove welds because these welds almost always extend the full length of the joint.

A short curved line below a square-groove symbol indicates weld contour. A short straight line in that position represents a flush weld surface. If the weld is not *to be* ground, however, that part of the symbol is usually omitted. When grinding is required, it must be indicated in the symbol:

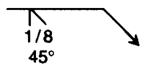
The root-opening size for a groove weld is written in within the symbol indicating the type of weld. For example, a ⅛-in root opening for a square-groove weld with a backing bar is specified by

Note that the “M” in the backing bar symbol indicates that the material to be used for backing is specified.

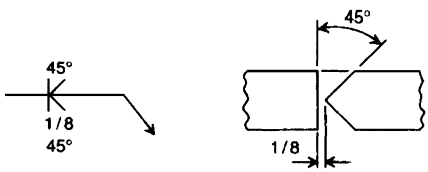
A ⅛-in root opening for a bevel weld, not to be ground, is indicated by

In this and other types of unsymmetrical welds, the arrow not only designates the arrow side of the joint but also points to the side to be shaped for the groove weld. When the arrow has this significance, the intention often is emphasized by an extra break in the arrow.

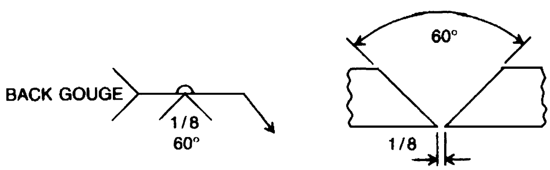
The angle at which the material is to be beveled should be indicated with the root opening:



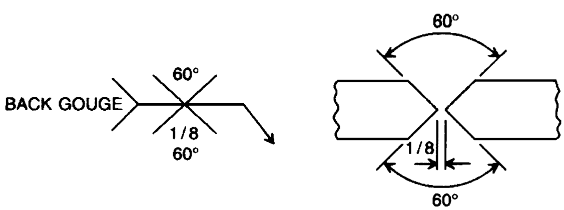
A double-bevel weld is specified by



A single-V weld is represented by



A double-V weld is indicated by



Summary. In preparing a weld symbol, insert size, weld-type symbol, length of weld, and spacing, in that order from left to right. The perpendicular leg of the symbol for fillet, bevel, J, and flare-bevel welds should be on the left of the symbol. Bear in mind also that arrow-side and otherside welds are the same size unless otherwise noted. When billing of detail material discloses the identity of the far side with the near side, the welding shown for the near side also will be duplicated on the far side. Symbols apply between abrupt changes in direction of welding unless governed by the all-around symbol or dimensioning shown.

Where groove preparation is not symmetrical and complete, additional information should be given on the symbol. Also it may be necessary to give weld-penetration information, as in Fig. 1.11. For the weld shown, penetration from either side must be a minimum of $\frac{3}{16}$ in. The second side should be back-gouged before the weld there is made.

Welds also may be a combination of different groove and fillet welds. While symbols can be developed for these, designers will save time by supplying a sketch or enlarged cross section. It is important to convey the required information accurately and completely to the workers who will do the job.

1.3.3 Welding material

Weldable structural steels permissible in buildings are listed in AISC Specification A3. Matching electrodes are given in AWS D1.1 Table 3.1.

1.3.4 Welding positions

The position of the stick electrode relative to the joint when a weld is being made *affects* welding economy and quality.

The basic welding positions are as follows:

Flat with the face of the weld nearly horizontal. The electrode is nearly vertical, and welding is performed from above the joint.

Horizontal with the axis of the weld horizontal. For groove welds, the face of the weld is nearly vertical. For fillet welds, the face of

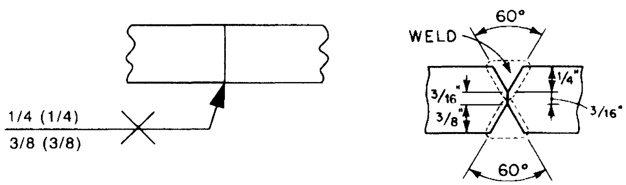


Figure 1.11 Penetration information is given on the welding symbol in (*a*) for the weld shown in (*b*). Penetration must be at least $\frac{3}{16}$ in. Second side must be back-gouged before the weld on that side is made.

the weld usually is about 45° relative to horizontal and vertical surfaces.

Vertical with the axis of the weld nearly vertical. (Welds are made upward.)

Overhead with the face of the weld nearly horizontal. The electrode is nearly vertical, and welding is performed from below the joint.

Where possible, welds should be made in the flat position. Weld metal can be deposited faster and more easily and generally the best and most economical welds are obtained. In a shop, the work usually is positioned to allow flat or horizontal welding. With care in design, the expense of this positioning can be kept to a minimum. In the field, vertical and overhead welding sometimes may be necessary. The best assurance of good welds in these positions is use of proper electrodes by experienced welders.

AWS D1.1 requires that only the flat position be used for submerged-arc welding, except for certain sizes of fillet welds. Single-pass fillet welds may be made in the flat or the horizontal position in sizes up to ⁵⁄₁₆ in with a single electrode and up to ½ in with multiple electrodes. Other positions are prohibited.

When groove-welded joints can be welded in the flat position, submerged-arc and gas metal-arc processes usually are more economical than the manual shielded metal-arc process.

Designers and detailers should detail connections to ensure that welders have ample space for positioning and manipulating electrodes and for observing the operation with a protective hood in place. Electrodes may be up to 18 in long and ⅜ in in diameter.

In addition, adequate space must be provided for deposition of the required size of the fillet weld. For example, to provide an adequate landing c, in, for the fillet weld of size D, in, in Fig. 1.12, c should be at least $D + \frac{5}{16}$. In building column splices, however, $c = D + \frac{3}{16}$ often is used for welding splice plates to fillers.

1.3.5 Weld procedures

Welds should be qualified and should be made only by welders, welding operators, and tackers qualified as required in AWS D1.1 for buildings. Welding should not be permitted under any of the following conditions:

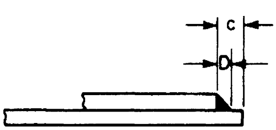


Figure 1.12 Minimum landing for a fillet weld.

When the ambient temperature is below 0°F

When surfaces are wet or exposed to rain, snow, or high wind

When welders are exposed to inclement conditions

Surfaces and edges to be welded should be free from fins, tears, cracks, and other defects. Also, surfaces at and near welds should be free from loose scale, slag, rust, grease, moisture, and other material that may prevent proper welding. AWS specifications, however, permit mill scale that withstands vigorous wire brushing, a light film of drying oil, or antispatter compound to remain. But the specifications require all mill scale to be removed from surfaces on which flange-to-web welds are to be made by submerged-arc welding or shielded metal-arc welding with low-hydrogen electrodes.

Parts to be fillet-welded should be in close contact. The gap between parts should not exceed 3⁄16 in. If it is more than 1⁄16 in, the fillet weld size should be increased by the amount of separation. The separation between faying surfaces for plug and slot welds and for butt joints landing on a backing should not exceed 1⁄16 in. Parts to be joined at butt joints should be carefully aligned. Where the parts are effectively restrained against bending due to eccentricity in alignment, an offset not exceeding 10 percent of the thickness of the thinner part joined, but in no case more than 1⁄8 in, is permitted as a departure from theoretical alignment. When correcting misalignment in such cases, the parts should not be drawn in to a greater slope than 1⁄2 in in 12 in.

For permissible welding positions, see Sec 1.3.4. Work should be positioned for flat welding whenever practicable.

In general, welding procedures and sequences should avoid needless distortion and should minimize shrinkage stresses. As welding progresses, welds should be deposited so as to balance the applied heat. Welding of a member should progress from points where parts are relatively fixed in position toward points where parts have greater relative freedom of movement. Where it is impossible to avoid high residual stresses in the closing welds of a rigid assembly, these welds should be made in compression elements. Joints expected to have significant shrinkage should be welded before joints expected to have lesser shrinkage, and restraint should be kept to a minimum. If severe external restraint against shrinkage is present, welding should be carried continuously to completion or to a point that will ensure freedom from cracking before the joint is allowed to cool below the minimum specified preheat and interpass temperatures.

In shop fabrication of cover-plated beams and built-up members, each component requiring splices should be spliced before it is welded to other parts of the member. Up to three subsections may be spliced to form a long girder or girder section.

With too rapid cooling, cracks might form in a weld. Possible causes are shrinkage of weld and heat-affected zone, austenite-martensite transformation, and entrapped hydrogen. Preheating the base metal can eliminate the first two causes. Preheating reduces the temperature gradient between weld and adjacent base metal, thus decreasing the cooling rate and resulting stresses. Also, if hydrogen is present, preheating allows more time for this gas to escape. Use of low-hydrogen electrodes, with suitable moisture control, is also advantageous in controlling hydrogen content.

High cooling rates occur at arc strikes that do not deposit weld metal. Hence strikes outside the area of permanent welds should be avoided. Cracks or blemishes resulting from arc strikes should be ground to a smooth contour and checked for soundness.

To avoid cracks and for other reasons, AWS specifications require that under certain conditions, before a weld is made the base metal must be preheated. Table 1.10 lists typical preheat and interpass

TABLE 1.10 Requirements of AWS D1.1 for Minimum Preheat and Interpass Temperatures, °F, for Welds in Buildings for Some Commonly Used Structural Steels*

Thickness at thickest part at point of welding, in	Shielded metal-arc with other than low-hydrogen electrodes **ASTM A36, A53 grade B, A501, A529**	Shielded metal-arc with low-hydrogen electrodes; submerged-arc, gas metal-arc, or flux-cored arc **ASTM A36, A53 grade B, A441, A501, A529 grades 50 and 55, A572 grades 42, 50, and 55, A588, A992**	Shielded metal-arc with low-hydrogen electrodes; submerged-arc, gas metal-arc, or flux-cored arc **ASTM A572 grade 60 and 65**	Shielded metal-arc; submerged-arc, gas metal-arc, or flux-cored arc with electrodes or electrode-flux combination capable of depositing weld metal with a maximum diffusible hydrogen content of 8 mL/100 g when tested in accordance with AWS A4.3 **ASTM A913‡ grades 50, 60, and 65**
To ¾	32†	32†	50	32†
Over ¾ to 1½	150	50	150	32†
1½ to 2½	225	150	225	32†
Over 2½	300	225	300	32†

*In joints involving different base metals, preheat as specified for higher-strength base metal.

†When the base-metal temperature is below 32°F, the base metal shall be preheated to at least 70°F and the minimum interpass temperature shall be maintained during welding.

‡The heat input limitations of AWS D1.1 paragraph 5.7 shall not apply to A913.

temperatures. The table recognizes that as plate thickness, carbon content, or alloy content increases, higher preheats are necessary to lower cooling rates and to avoid microcracks or brittle heat-affected zones.

Preheating should bring to the specified preheat temperature the surface of the base metal within a distance equal to the thickness of the part being welded, but not less than 3 in of the point of welding. This temperature should be maintained as a minimum interpass temperature while welding progresses.

Preheat and interpass temperatures should be sufficient to prevent crack formation. Temperatures above the minimums in Table 1.10 may be required for highly restrained welds.

Peening sometimes is used on intermediate weld layers for control of shrinkage stresses in thick welds to prevent cracking. It should be done with a round-nose tool and light blows from a power hammer after the weld has cooled to a temperature warm to the hand. The root or surface layer of the weld or the base metal at the edges of the weld should not be peened. Care should be taken to prevent scaling or flaking of weld and base metal from overpeening.

When required by plans and specifications, welded assemblies should be stress-relieved by heat treating. (See AWS D1.1 for temperatures and holding times required.) Finish machining should be done after stress relieving.

Tack and other temporary welds are subject to the same quality requirements as final welds. For tack welds, however, preheat is not mandatory for single-pass welds that are remelted and incorporated into continuous submerged-arc welds. Also, defects such as undercut, unfilled craters, and porosity need not be removed before final submerged-arc welding. Welds not incorporated into final welds should be removed after they have served their purpose, and the surface should be made flush with the original surface.

Before a weld is made over previously deposited weld metal, all slag should be removed, and the weld and adjacent material should be brushed clean.

Groove welds should be terminated at the ends of a joint in a manner that will ensure sound welds. Where possible, this should be done with the aid of weld tabs or runoff plates. AWS D1.1 does not require removal of weld tabs for statically loaded structures but does require it for dynamically loaded structures. The AISC Seismic Provisions (2005) also require their removal in zones of high seismicity. The ends of the welds then should be made smooth and flush with the edges of the abutting parts.

After welds have been completed, slag should be removed from them. The metal should not be painted until all welded joints have been completed, inspected, and accepted. Before paint is applied, spatter, rust, loose scale, oil, and dirt should be removed.

AWS D1.1 presents details of techniques acceptable for welding buildings. These techniques include handling of electrodes and fluxes.

1.3.6 Weld quality

A basic requirement of all welds is thorough fusion of weld and base metal and of successive layers of weld metal. In addition, welds should not be handicapped by craters, undercutting, overlap, porosity, or cracks. (AWS D1.1 gives acceptable tolerances for these defects.) If craters, excessive concavity, or undersized welds occur in the effective length of a weld, they should be cleaned and filled to the full cross section of the weld. Generally, all undercutting (removal of base metal at the toe of a weld) should be repaired by depositing weld metal to restore the original surface. Overlap (a rolling over of the weld surface with lack of fusion at an edge), which may cause stress concentrations, and excessive convexity should be reduced by grinding away of excess material (see Figs. 1.13 and 1.14). If excessive porosity, excessive slag inclusions, or incomplete fusion occur, the defective portions should be removed and rewelded. If cracks are present, their extent should be determined by acid etching, magnetic-particle inspection, or other equally positive means. Not only the cracks but also sound metal 2 in beyond their ends should be removed and replaced with the weld metal. Use of a small electrode for this purpose reduces the chances of further defects due to shrinkage. An electrode not more than 5⁄32 in in

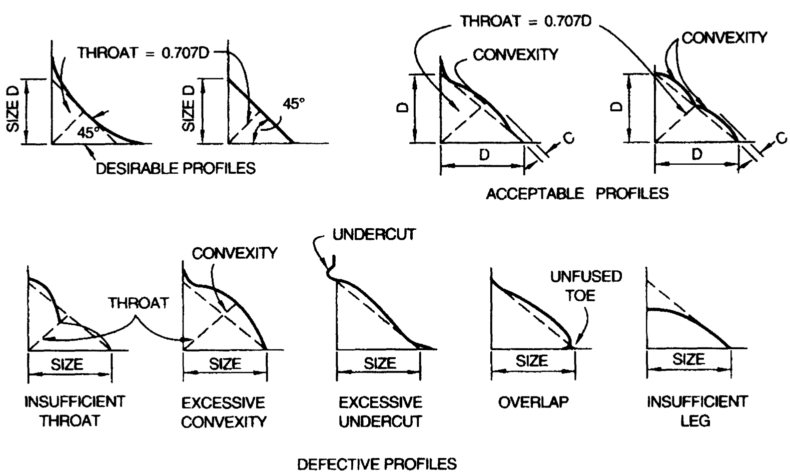


Figure 1.13 Profiles of fillet welds.

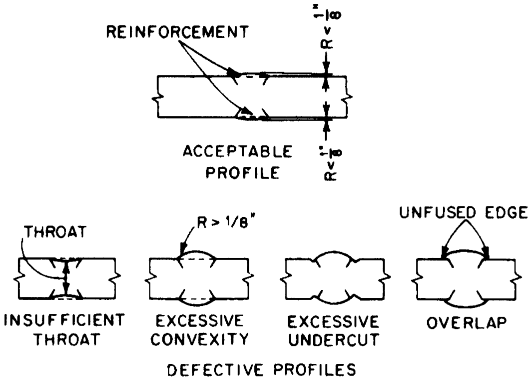


Figure 1.14 Profiles as groove welds.

diameter is desirable for depositing weld metal to compensate for size deficiencies.

AWS D1.1 limits convexity C to the values in Table 1.11.

Weld-quality requirements should depend on the job the welds are to do. Excessive requirements are uneconomical. Size, length, and penetration are always important for a stress-carrying weld and should completely meet design requirements. Undercutting, on the other hand, should not be permitted in main connections, such as those in trusses and bracing, but small amounts might be permitted in less important connections, such as those in platform framing for an industrial building. Type of electrode, similarly, is important for stress-carrying welds but not so critical for many miscellaneous welds. Again, poor appearance of a weld is objectionable if it indicates a bad weld or if the weld will be exposed where aesthetics is a design consideration, but for many types of structures, such as factories, warehouses, and incinerators, the appearance of a good weld is not critical. A sound weld is important, but a weld entirely free of porosity or small slag inclusions should be required only when the type of loading actually requires this perfection.

Welds may be inspected by one or more methods: visual inspection; nondestructive tests, such as ultrasonic, x-ray, dye penetration, magnetic

TABLE 1.11 AWS D1.1 Limits on Convexity of Fillet Welds

Measured leg size or width of surface bead, in	Maximum convexity, in
$\frac{5}{16}$ or less	$\frac{1}{16}$
Over $\frac{5}{16}$ but less than 1	$\frac{1}{8}$
1 or more	$\frac{3}{16}$

particle, and cutting of samples from finished welds. Designers should specify which welds are to be examined, extent of the examination, and methods to be used.

1.3.7 Methods for determining strength of skewed fillet welds

It is often beneficial to utilize skewed single-plate or end-plate shear connections to carry members which run nonorthogonal to their supports. In such case the welds attaching the connection material to the support must be designed to accommodate this skew. There are two ways to do this. The AWS D1.1 Structural Welding Code provides a method to calculate the effective throat for skewed T joints with varying dihedral angles, which is based on providing equal strength in the obtuse and acute welds. This is shown in Fig. 1.15*a*. The AISC method is simpler, and simply increases the weld size on the obtuse side by the amount of the gap, as is shown in Fig. 1.15*c*.

Both methods can be shown to provide a strength equal to or greater than the required orthogonal weld size of *W*. The main difference with regard to strength is that the AWS method, as given by the formulas in Fig. 1.16, maintains equal strength in both fillets, whereas the AISC method increases the strength on the acute side by maintaining a constant fillet size, $W_a = W$, while the increased size, $W_o = W + g$, on the obtuse side actually loses strength because of the gap, *g*. Nevertheless, it can be shown that the sum of the strengths of these two fillet welds, $W_a = W$ and $W_o = W + g$, is always greater than the $2W$ of the required orthogonal fillets.

It should be noted that the gap, *g*, is limited to a maximum value of ³⁄₁₆ in for both methods.

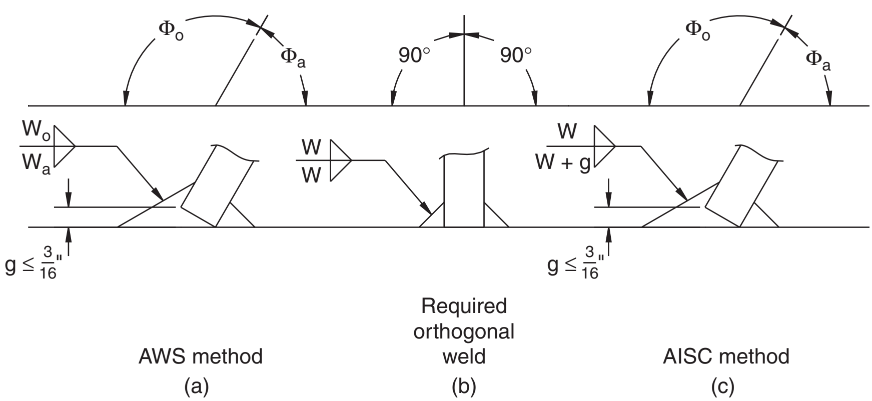


Figure 1.15 Skewed fillet weld sizes required to match strength of required orthogonal fillets of size *W*.

The effects of the skew on the effective throat of a fillet weld can be very significant as shown in Fig. 1.16. Figure 1.16 also shows how fillet legs W_o and W_a are measured in the skewed configuration. On the acute side of the connection the effective throat for a given fillet weld size gradually increases as the connection intersection angle, Φ, changes from 90 to 60°. From 60 to 30°, the weld changes from a fillet weld to a partial joint

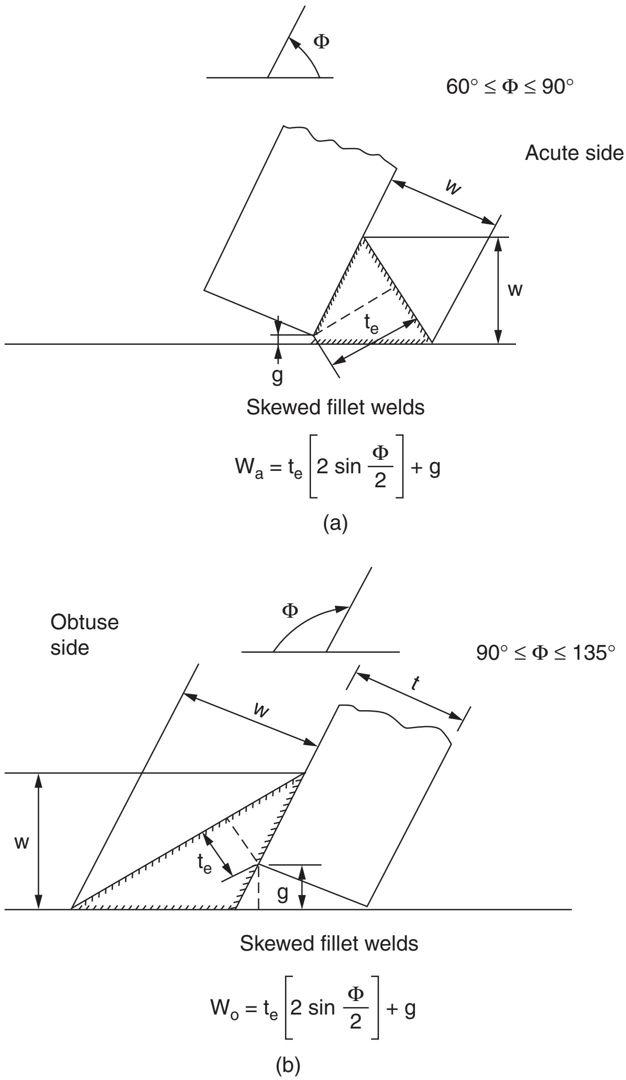


Figure 1.16 Geometry of skewed fillet welds. (Relationship of weld size to effective throat, t_e) (*a*) acute side (*b*) obtuse side. Note how the skewed fillet welds are to be measured. The contact leg length is *not* the weld size.

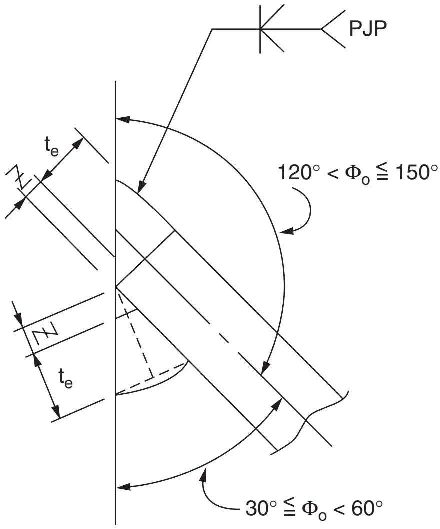


Figure1.17 Acute angles less than 60° and obtuse angles greater than 120°.

penetration (PJP) groove weld (Fig. 1.17) and the effective throat, t_e, decreases due to the allowance, z, for the unwelded portion at the root. While this allowance varies based on the welding process and position, it can conservatively be taken as the throat less ⅛ in for 60 to 45° and less ¼ in for 45 to 30°. Joints less than 30° are not prequalified and generally should not be used.

1.3.8 Obliquely loaded concentric fillet weld groups

The strength of a fillet weld is dependent on the direction of loading. Welds that are loaded in their longitudinal direction have a design strength of $0.6F_{EXX}$, while welds loaded transverse to their longitudinal axis have a design strength 1.5 times greater. The strength of welds loaded between these extremes can be found as

$$F_w = 0.6F_{EXX}(1.0 + 0.50 \sin^{1.5} \theta)$$

This equation is easily applied to a single-line weld, or a group of parallel-line welds, but when applied to weld groups containing welds loaded at differing angles, such as that given in Fig. 1.18, its application becomes much more complex. In such cases, deformation compatibility must also be satisfied. Since the transversely loaded welds are considerably less ductile than the longitudinally loaded welds, the transversely loaded welds will fracture before the longitudinally loaded welds reach their full capacity. This can easily be seen by examining Fig. 1.19 (taken from Fig. 8-5 AISC 2005). A weld loaded transverse to its longitudinal direction will fracture at a deformation equal to approximately 0.056

times the weld size. At this same deformation the longitudinally loaded weld has only reached about 83 percent of its maximum strength.

To account for this the strength of the weld is calculated as

$$R_{nx} = \sum F_{wix} A_{wi}$$
$$R_{ny} = \sum F_{wiy} A_{wi}$$

where A_{wi} = effective area of weld throat of any ith weld element, in²

$F_{wi} = 0.6F_{EXX}(1 + 0.50\sin^{1.5}\theta)f(p)$

F_{wi} = nominal stress in any ith weld element, ksi

F_{wix} = x component of stress F_{wi}

F_{wiy} = y component of stress F_{wi}

$f(p) = [p(1.9 - 0.9p)]^{0.3}$

$p = \Delta/\Delta_m$, ratio of element i deformation to its deformation at maximum stress

$\Delta_m = 0.209(\theta + 2)^{-0.32}w$, deformation of weld element at maximum stress, in (mm)

Δ_i = deformation of weld elements at intermediate *stress* levels, linearly proportioned to the critical deformation based on distance from the instantaneous center of rotation, r_i, in $= \Delta_i = r_i\Delta_u/r_{crit}$

$\Delta_u = 1.087(\theta + 6)^{-0.65}w \le 0.17w$, deformation of weld element at ultimate stress (fracture), usually in element furthest from instantaneous center of rotation, in (mm)

w = leg size of the fillet weld, in

r_{crit} = distance from instantaneous center of rotation to weld element with minimum Δ_u/r_i ratio, in

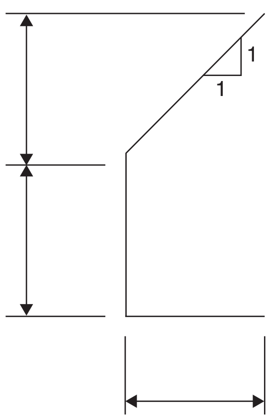


Figure 1.18 Obliquely loaded weld group.

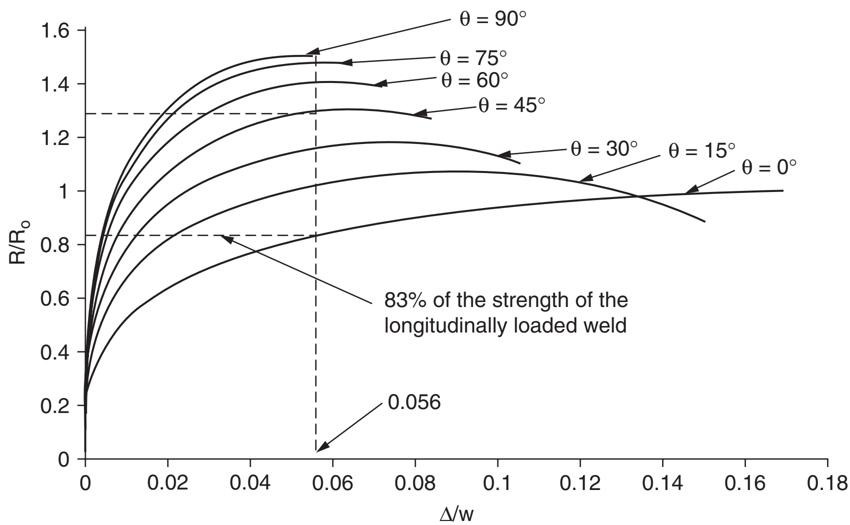


Figure 1.19 Graphical solution of the capacity of an obliquely loaded weld group. Alternately, if the welds are loaded only in the transverse and longitudinal directions, then the weld strength is permitted to taken as the greater of $R_n = R_{wl} + R_{wt}$ or $R_n = 0.85\,R_{wl} + 1.5\,R_{wt}$.

This can be accomplished graphically using Fig. 1.19, the load-deformation curves. For example, to find the strength of the concentrically loaded weld group shown in Fig. 1.18, first the least ductile weld is determined. In this case it is the transversely loaded weld. By drawing a vertical line from the point of fracture, the strength increase or decrease for the remaining elements can be determined. In this case the strength of the weld group of Fig. 1.18, with I = 1m is found to be

$$\phi R_w = (D)(1.392)(1.5(1) + 1.29(1.41) + 0.83(1)) = 5.78D$$

1.4 References

1. American Institute of Steel Construction, *Manual of Steel Construction*, 13th ed, Chicago, IL, 2005.
2. American Welding Society, *Structural Welding Code*, D1.1, Miami, FL, 2006.
3. Research Council on Structural Connections, “Specification for Structural Joints Using ASTM A325 or A490 Bolts,” American Institute of Steel Construction, Chicago, IL, 2004.
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Chapter

2

Design of Connections for Axial, Moment, and Shear Forces

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(*Courtesy of The Steel Institute of New York.*)

2.1 Introduction

Connection design is an interesting subject because it requires a great deal of rational analysis in arriving at a solution. There are literally an infinite number of possible connection configurations, and only a very small number of these have been subjected to physical testing. Even within the small group that has been tested, changes in load directions, geometry, material types, fastener type, and arrangement very quickly result in configurations that have not been tested and therefore require judgment and rational analysis on the part of the designer. This chapter provides design approaches to connections based on test data, when available, supplemented by rational design or art and science in the form of equilibrium (admissible force states), limit states, and ductility considerations. The limit states are those of the AISC 13th Edition Manual (2005).

2.1.1 Philosophy

Connection design is both an art and a science. The science involves equilibrium, limit states, load paths, and the lower bound theorem of limit analysis. The art involves the determination of the most efficient load paths for the connection, and this is necessary because most connections are statically indeterminate.

The lower bound theorem of limit analysis states: If a distribution of forces within a structure (or connection, which is a localized structure) can be found, which is in equilibrium with the external load and which satisfies the limit states, then the externally applied load is less than or at most equal to the load that would cause connection failure. In other words, any solution for a connection that satisfies equilibrium and the limit states yields a safe connection. This is the science of connection design. The art involves finding the internal force distribution (or load paths) that maximizes the external load at which a connection fails. This maximized external load is also the true failure load when the internal force distribution results in satisfaction of compatibility (no gaps and tears) within the connection in addition to satisfying equilibrium and the limit states.

It should be noted that, strictly speaking, the lower bound theorem applies only to yield limit states in structures that are ductile. Therefore, in applying it to connections, limit states involving stability and fracture (lack of ductility) must be considered to preclude these modes of failure.

2.1.2 General procedure

Determine the external (applied) loads, also called required strengths, and their lines of action. Make a preliminary layout, preferably to scale. The connection should be as compact as possible to conserve material and to minimize interferences with utilities, equipment, and access, and to facilitate shipping and handling. Decide on where bolts and welds will be used and select bolt type and size. Decide on a load path through the connection. For a statically determinate connection, there is only one possibility, but for indeterminate connections, there are many possibilities. Use judgment, experience, and published information to arrive at the best load path. Now provide sufficient strength, stiffness, and ductility, using the limit states identified for each part of the load path, to give the connection sufficient design strength, that is, to make the connection adequate to carry the given loads. Complete the preliminary layout, check specification-required spacings, and finally check to ensure that the connection can be fabricated and erected. The examples of this chapter will demonstrate this procedure.

2.1.3 Economic considerations

For any given connection situation, it is usually possible to arrive at more than one satisfactory solution. Where there is a possibility of using bolts or welds, let the economics of fabrication and erection play a role in the choice. Different fabricators and erectors in different parts of the country have their preferred ways of working, and as long as the principles of connection design are followed to achieve a safe connection, local preferences should be accepted. Some additional considerations that will result in more economical connections (Thornton, 1995B) are:

1. For shear connections, provide the actual loads and allow the use of single plate and single angle shear connections. Do not specify full-depth connections or rely on the AISC uniform load tables.
2. For moment connections, provide the actual moments and the actual shears. Also, provide a "breakdown" of the total moment, that is, give the gravity moment and lateral moment due to wind or seismic loads separately. This is needed to do a proper check for column web doubler plates. If stiffeners are required, allow the use of fillet welds in place of complete joint penetration welds. To avoid the use of stiffeners, consider redesigning with a heavier column to eliminate them.
3. For bracing connections, in addition to providing the brace force, also provide the beam shear and axial transfer force. The transfer force is the axial force that must be transferred to the opposite side of the column. The transfer force is not necessarily the beam axial force that is obtained from a computer analysis of the structure. See Thornton (1995B) and Thornton and Muir (2008) for a discussion of this. A misunderstanding of transfer forces can lead to both uneconomic and unsafe connections.

2.1.4 Types of connections

There are three basic forces to which connections are subjected. These are axial force, shear force, and moment. Many connections are subject to two or more of these simultaneously. Connections are usually classified according to the major load type to be carried, such as shear connections, which carry primarily shear; moment connections, which carry primarily moment; and axial force connections, such as splices, bracing and truss connections, and hangers, which carry primarily axial force. Subsequent sections of this chapter will deal with these three basic types of connections.

2.1.5 Organization

This chapter will cover axial force connections first, then moment connections, and lastly shear connections. This is done to emphasize the ideas of load paths, limit states, and the lower bound theorem, which (except for limit states) are less obviously necessary to consider for the simpler connections.

This chapter is based on the limit states of the AISC LRFD Specification (AISC, 2005). The determination of loads, that is, required strengths, is dependent upon the specific building code required for the project, based on location, local laws, and so forth. At this time (2008), there is much transition taking place in the determination of seismic loads and connection requirements. Wherever examples involving seismic loads are presented in this chapter, the solutions presented are indicative of the author's experience in current practice with many structural engineers, and may need to be supplemented with additional requirements from local seismic codes. Chapter 5 deals with connections in high seismic regions and covers these additional requirements.

2.2 Axial Force Connections

2.2.1 Bracing connections

2.2.1.1 Introduction. The lateral force-resisting system in buildings may consist of a vertical truss. This is referred to as a braced frame and the connections of the diagonal braces to the beams and columns are the bracing connections. Figure 2.1 shows various bracing arrangements. For the bracing system to be a true truss, the bracing connections should be concentric, that is, the gravity axes of all members at any joint should intersect at a single point. If the gravity axes are not concentric, the resulting couples must be considered in the design of the members. The examples of this section will be of concentric type, but the nonconcentric type can also be handled as will be shown.

2.2.1.2 Example 1. Consider the bracing connection of Fig. 2.2. The brace load is 855 kips, the beam shear is 10 kips, and the beam axial force is 411 kips. The horizontal component of the brace force is 627 kips, which means that $627 - 411 = 216$ kips is transferred to the opposite side of the column from the brace side. There must be a connection on this side to "pick up" this load, that is, provide a load path.

The design of this connection involves the design of four separate connections. These are (1) the brace-to-gusset connection, (2) the gusset-to-column connection, (3) the gusset-to-beam connection, and (4) the beam-to-column connection. A fifth connection is the connection on the other side of the column, which will not be considered here.

1. *Brace-to-gusset*: This part of the connection is designed first because it provides a minimum size for the gusset plate which is then used to design the gusset-to-column and gusset-to-beam connections. Providing an adequate load path involves the following limit states:

   a. Bolts (A325SC-B-N 1 $^1/_8$-in-diameter standard holes, serviceability limit state): The above notation indicates that the bolts are

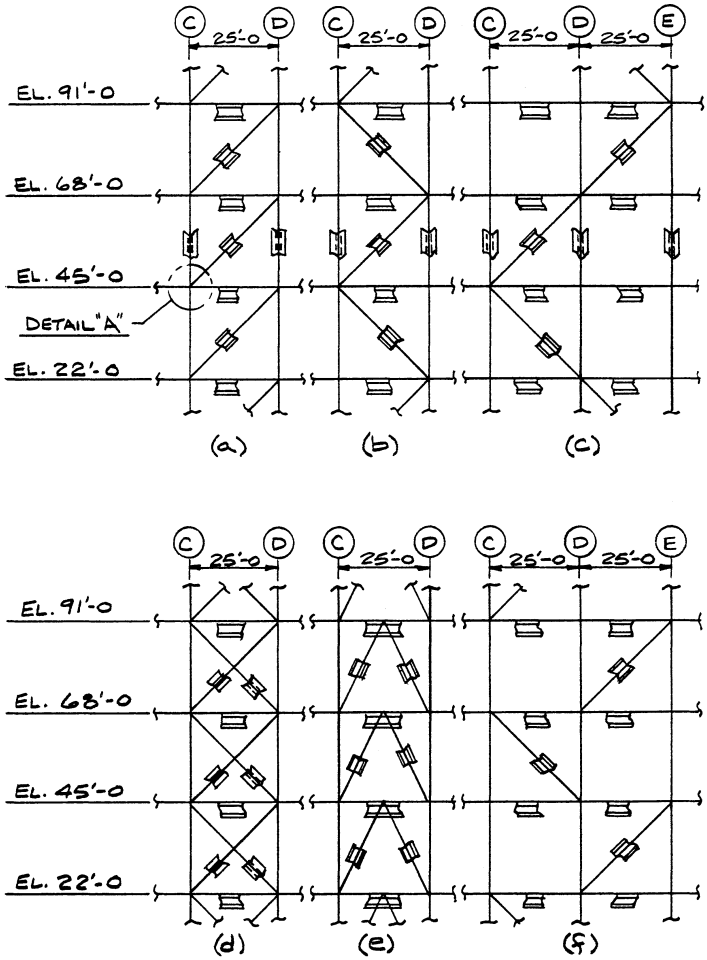


Figure 2.1 Various vertical bracing arrangements.

slip critical, the surface class is B, and threads are not excluded from the shear planes. The slip-critical design strength per bolt is

$$\phi r_{\text{str}} = 1 \times 1.13 \times 0.5 \times 56 = 31.6 \text{ kips}$$

The specification requires that connections designed as slip critical must also be checked as bearing for the bearing condition. The bearing design strength per bolt is

$$\phi r_v = 0.75 \times \frac{\pi}{4} \times 1.125^2 \times 48 = 35.8 \text{ kips}$$

Since 31.6 < 35.8, use 31.6 kips as the design strength. The estimated number of bolts required is 855/(31.6 × 2) = 13.5. Therefore, try 14 bolts each side of the connection.

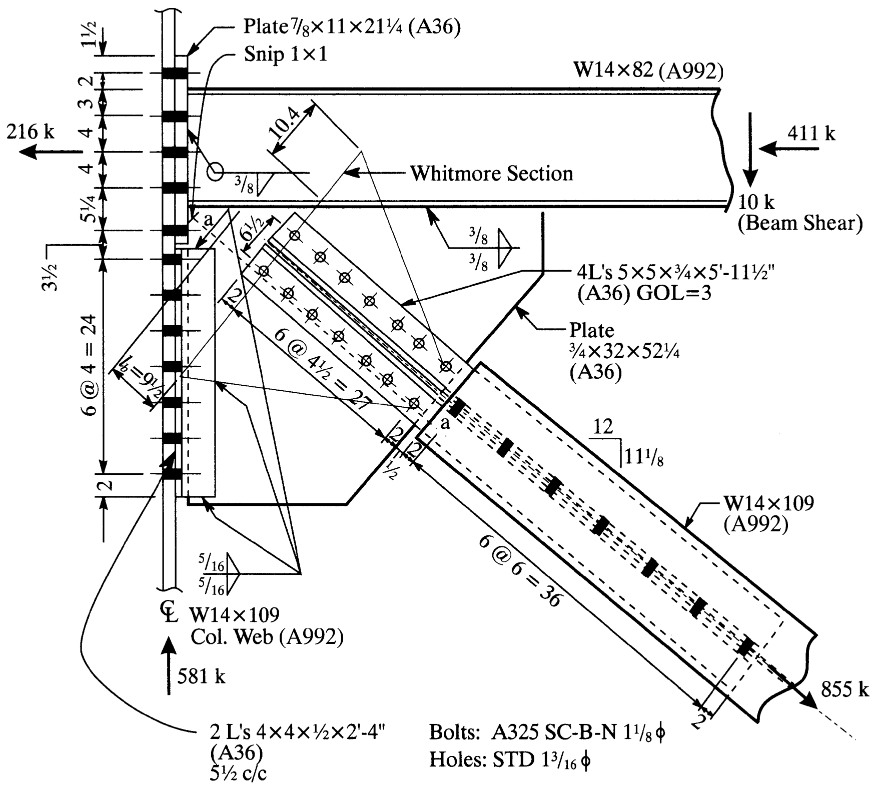


Figure 2.2 Example 1, bracing connection design.

b. W14 × 109 *brace checks:*

(1) *Bolt shear, bearing, and tearout*: The 1999 AISC Specification included for the first time a limit state that considers a bolt tearing out through the edge of the connected material. This requirement is carried forward into the 2005 AISC Specification. The proper check is one that considers bolt shear, bearing, and tearout for each bolt individually. The resistances of the individual bolts are then summed to determine a capacity for the bolt group.

The bolt shear strength has already been established as 31.6 kips per bolt.

The bearing strength per bolt is

$$\phi r_p = 0.75 \times 2.4 \times 1.125 \times 0.525 \times 65 = 69.1 \text{ kips}$$

The bolt tearout capacity of the edge bolts at the brace web is

$$\phi r_p = 0.75 \times 1.2 \times (2 - 0.594) \times 0.525 \times 65 = 43.1 \text{ kips}$$

Since tearout through the edge of the brace web is the critical condition and results in a capacity greater than the shear strength of the bolt, the full bearing capacity of the bolt can be developed. However, since the connection is to be designed as slip critical, the slip resistance will govern.

(2) *Block shear rupture:*

$$A_{gv} = (2 + 6 \times 6) \times 0.525 \times 2 = 39.9 \text{ in}^2$$
$$A_{nv} = 39.9 - 6.5 \times 1.25 \times 0.525 \times 2 = 31.4 \text{ in}^2$$
$$A_{nt} = 3\ .54 - 1 \times 1.25 \times 0.525 = 2.88 \text{ in}^2$$

Shear yielding = 39.9 × 0.6 × 50 = 1200 kips
Shear fracture = 31.4 × 0.6 × 65 = 1220 kips
Tension fracture = 2.88 × 65 = 187 kips

Since shear yielding is less than shear fracture, the failure mode is shear yielding and tension fracture; thus, the design block shear strength is

$$\phi R_{bs} = 0.75(1200 + 187) = 1040 \text{ kips} > 855 \text{ kips, ok}$$

c. *Gusset checks:*

(1) *Bearing and tearout:* The bearing strength per bolt is

$$\phi r_p = 0.75 \times 2.4 \times 1.125 \times 0.75 \times 58 = 88.1 \text{ kips}$$

The bolt tearout capacity of the edge bolts at the gusset is

$$\phi r_p = 0.75 \times 1.2 \times (2 - 0.594) \times 0.75 \times 58 = 55.0 \text{ kips}$$

Again the bolt shear governs.

(2) *Block shear rupture:* These calculations are similar to those for the brace.

$$\begin{aligned}
A_{gv} &= 29.0 \times 0.75 \times 2 = 43.5 \text{ in}^2\\
A_{nv} &= (29.0 - 6.5 \times 1.25) \times 0.75 \times 2 = 31.3 \text{ in}^2\\
A_{nt} &= (6.5 - 1.0 \times 1.25) \times 0.75 = 3.94 \text{ in}^2\\
\phi R_{bs} &= 0.75\,[F_u A_{nt} + \min\{0.6\,F_y A_{gv},\ 0.6\,F_u A_{nv}\}]\\
&= 0.75\,[58 \times 3.94 + \min\{0.6 \times 36 \times 43.5,\\
&\qquad 0.6 \times 58 \times 31.3\}]\\
&= 0.75\,[229 + \min\{940,\ 1090\}]\\
&= 876 \text{ kips} > 855 \text{ kips ok}
\end{aligned}$$

(3) *Whitmore section:* Since the brace load can be compression, this check is used to check for gusset buckling. Figure 2.2 shows the "Whitmore section" length, which is normally $l_w = (27 \tan 30) \times 2 + 6.5 = 37.7$ in, but the section passes out of the gusset and into the beam web at its upper side. Because of the fillet weld of the

gusset to the beam flange, this part of the Whitmore section is not ineffective, that is, load can be passed through the weld to be carried on this part of the Whitmore section. The effective length of the Whitmore section is thus

$$l_{\text{we}} = (37.7 - 10.4) + 10.4 \times \frac{0.510}{0.75} \times \frac{50}{36} = 27.3 + 9.8 = 37.1 \text{ in}$$

The gusset buckling length is, from Fig. 2.1, $l_b = 9.5$ in, and the slenderness ratio is

$$\frac{Kl_b}{r} = \frac{0.5 \times 9.5 \times \sqrt{12}}{0.75} = 21.9$$

In this formula, the theoretical fixed-fixed factor of 0.5 is used rather than the usually recommended value of 0.65 for columns, because of the conservatism of this buckling check as determined by Gross (1990) from full-scale tests. From the AISC 2005 Specification Section J4.4, since $Kl_b/r \leq 25$, the design buckling strength is

$$\phi F_{cr} = 0.9 \times 36 = 32.4 \text{ ksi}$$

and the Whitmore section buckling strength is thus

$$\phi R_{wb} = 32.4 \times 37.1 \times 0.75 = 902 \text{ kips} > 855 \text{ kips, ok}$$

d. Brace-to-gusset connection angles:

(1) *Gross and net area*: The gross area required is $855/(0.9 \times 36) = 26.4 \text{ in}^2$
Try 4 *Ls* $5 \times 5 \times 3/4$, $A_{gt} = 6.94 \times 4 = 27.8 \text{ in}^2$, ok
The net area is $A_{nt} = 27.8 - 4 \times 0.75 \times 1.25 = 24.1 \text{ in}^2$
The effective net area is the lesser of $0.85\,A_{gt}$ or UA_{nt}, where $U = 1 - \dfrac{1.51}{27} = 0.944$. Thus $0.85\,A_{gt} = 0.85 \times 27.8 = 23.6$ and $UA_{nt} = 0.944 \times 24.1 = 22.8$ and then $A_e = 22.8$. Therefore, the net tensile design strength is $\phi R_t = 0.75 \times 58 \times 22.8 = 992 \text{ kips} > 855$ kips ok.

(2) *Bearing and tearout*: Comparing the strength of two $3/4''$ angles to the $3/4''$ gusset, it is clear that bolt bearing and tearout on the angles will not control.

(3) *Block shear rupture*: The length of the connection on the gusset side is the shorter of the two and is, therefore, the more critical. Per angle,

$$A_{gv} = 29.0 \times 0.75 = 21.75 \text{ in}^2$$
$$A_{nv} = (29.0 - 6.5 \times 1.25) \times 0.75 = 15.66 \text{ in}^2$$
$$A_{nt} = (2.0 - 0.5 \times 1.25) \times 0.75 = 1.03 \text{ in}^2$$

$$
\begin{aligned}
\phi R_{bs} &= 0.75\,[F_u A_{nt} + \min\{0.6\,F_y A_{gv},\ 0.6\,F_u A_{nv}\}] \\
&= 0.75\,[58 \times 1.03 + \min\{0.6 \times 36 \times 21.75, \\
&\quad 0.6 \times 58 \times 15.66\}] \times 4 \\
&= 1590 \text{ kips} > 855 \text{ kips ok}
\end{aligned}
$$

This completes the design checks for the brace-to-gusset connection. All elements of the load path, which consists of the bolts, the brace web, the gusset, and the connection angles, have been checked. The remaining connection interfaces require a method to determine the forces on them. Research (Thornton 1991, 1995b) and practice (AISC, 2005) have shown that the best method for doing this is the uniform force method (UFM). The force distributions for this method are shown in Fig. 2.3.

From the design of the brace-to-gusset connection, a certain minimum size of gusset is required. This is the gusset shown in Fig. 2.2. Usually, this gusset size, which is a preliminary size, is sufficient for the final design. From Figs. 2.2 and 2.3, the basic data are

$$\tan\theta = \frac{12}{11.125} = 1.08$$

$$e_B = \frac{14.3}{2} = 7.15$$

$$e_c = 0$$

The quantities α and β locate the centroids of the gusset edge connections, and in order for no couples to exist on

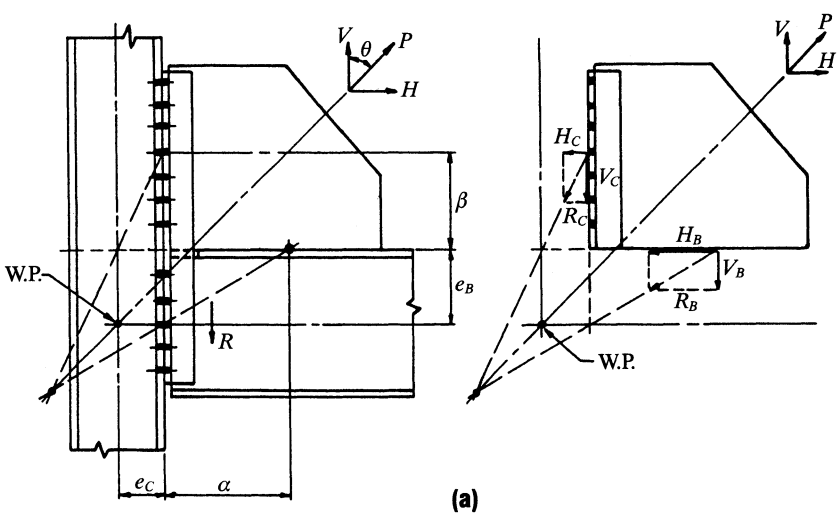
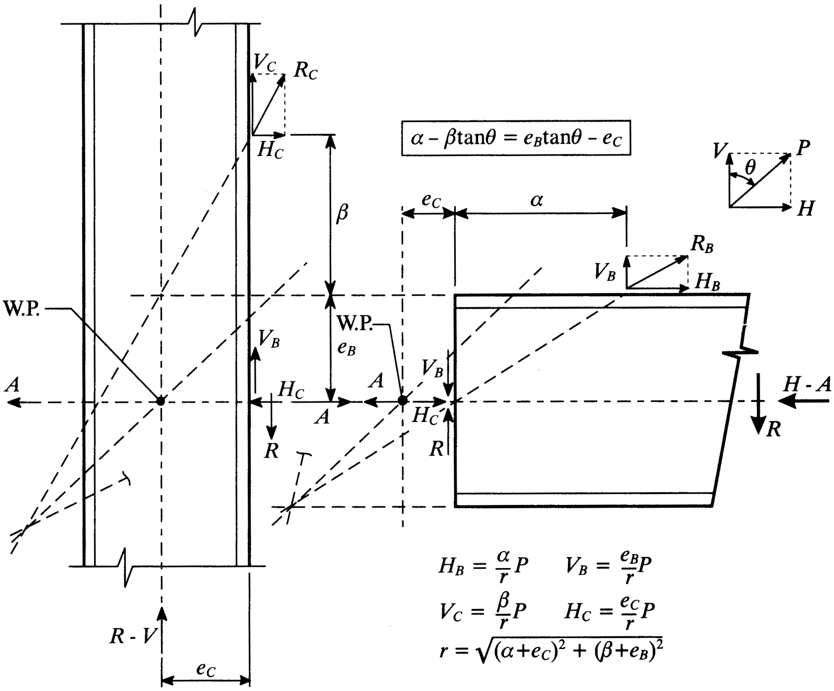


Figure 2.3*a* The uniform force method.



(b)

Figure 2.3*b* Force distribution for the uniform force method.

these connections, α and β must satisfy the following relationship given in Fig. 2.3*b*,

$$\alpha - \beta \tan \theta = e_b \tan \theta - e_c$$

Thus, $\alpha - 1.08\beta = 7.15 \times 1.08 - 0 = 7.72$.

From the geometry given in Fig. 2.2, a seven-row connection at 4-in pitch will give $\beta = 17.5$ in. Then $\alpha = 7.72 + 1.08 \times 17.5 = 26.6$ in and the horizontal length of the gusset is $(26.6 - 1) \times 2 = 51.2$ in. Choose a gusset length of $51\frac{1}{4}$ in.

With $\alpha = 26.6$ and $\beta = 17.5$,

$$r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2}$$
$$= \sqrt{(26.6 + 0)^2 + (17.5 + 7.15)^2} = 36.3 \text{ in}$$
$$V_c = \frac{\beta}{r}P = \frac{17.5}{36.3}855 = 413 \text{ kips}$$
$$H_c = \frac{e_c}{r}P = \frac{0}{36.3}855 = 0 \text{ kips}$$

$$H_B = \frac{\alpha}{r}P = \frac{26.2}{36.3}855 = 617\,\text{kips}$$
$$V_B = \frac{e_b}{r}P = \frac{7.15}{36.3}855 = 168\,\text{kips}$$

2. *Gusset-to-column*: The loads are 412 kips shear and 0 kips axial.

a. Bolts and clip angles:
Bolts: A325SC-B-N 1 $^1/_8$ ϕ; standard holes, serviceability criterion
Clip angles: try *Ls* 4 × 4 × $^1/_2$
Shear per bolt is

$$V = {}^{413}/_{14} = 29.5 \text{ kips} < 31.6 \text{ kips, ok}$$

The bearing strength of the clip angle is

$$\phi r_p = 0.75 \times 2.4 \times 58 \times 0.5 \times 1.125 = 58.7 \text{ kips} > 31.6 \text{ kips}$$

The bearing strength of the W14 × 109 column web is

$$\phi r_p = 0.75 \times 2.4 \times 65 \times 0.525 \times 1.125 = 69.1 \text{ kips} > 31.6 \text{ kips}$$

The bolt tearout capacity of the edge bolts at the clip angles is

$$\phi r_p = 0.75 \times 1.2 \times (2 - 0.594) \times 0.5 \times 58 = 36.7 \text{ kips} > 31.6 \text{ kips, ok}$$

There is no edge tearout condition at the column web, so it does not govern.
The net shear strength of the clips is

$$\begin{aligned}\phi R_n &= 0.75 \times 0.6 \times 58\,(28 - 7 \times 1.25) \times 0.5 \times 2 \\ &= 502 \text{ kips} > 412 \text{ kips, ok}\end{aligned}$$

The gross shear strength of the clips is

$$\phi R_n = 1.00 \times 0.6 \times 36 \times 28 \times 0.5 \times 2 = 605 \text{ kips} > 412 \text{ kips, ok}$$

Block shear on the clip angles

$$\begin{aligned}A_{gv} &= 26 \times 0.5 = 13.0 \text{ in}^2 \\ A_{nv} &= (26 - 6.5 \times 1.25) \times 0.5 = 8.94 \text{ in}^2 \\ A_{gt} &= (1.5 - 0.5 \times 1.25) \times 0.5 = 0.438 \text{ in}^2 \\ \phi R_{bs} &= 0.75\,[0.438 \times 58 + \min\{0.6 \times 36 \times 13.0,\ 0.6 \times 58 \times 8.94\}] \times 2 \\ &= 459 \text{ kips} > 412 \text{ kips, ok}\end{aligned}$$

b. Fillet weld of clip angles to gusset: The length of this clip angle weld is 28-in. From AISC 13th Edition Manual Table 8-8, l =28, kl = 3.0, k = 0.107, $al = 4 - xl = 4 - 0.009 \times 28 = 3.75$, and a = 0.134. By interpolation, c = 2.39, and the required fillet weld size is D = 412/(0.75 × 2.39 × 28 × 2) = 4.11, so the required fillet weld size is 5/16, and no proration is required because of the $^3/_4$-in thick gusset. (See Table 1.9 Chapter 1).

3. *Gusset-to-beam*: The loads are 627 kips shear and 168 kips axial. The length of the gusset is 52.25 in. The 1-in snip can be ignored with negligible effect on the stress.

   a. *Gusset stresses*:

$$f_v = \frac{627}{0.75 \times 51.25} = 16.3\,\text{ksi} < 1.0 \times 0.6 \times 36 = 21.6\,\text{ksi, ok}$$

$$f_a = \frac{168}{0.75 \times 51.25} = 4.37\,\text{ksi} < 0.9 \times 36 = 32.4\,\text{ksi, ok}$$

   b. *Weld of gusset to beam bottom flange*: The resultant force per inch of weld is

$$f_r = \sqrt{16.3^2 + 4.37^2} \times \frac{0.75}{2} = 6.33\,\text{kips in}$$

   To account for the directional strength increase on fillet welds

$$\phi = \tan^{-1}\left(\frac{4.37}{16.3}\right) = 15.0°$$

$$\mu = 1.0 + 0.5\sin^{1.5}\phi = 1.0 + 0.5\sin^{1.5}(15) = 1.07$$

   The required weld size is

$$D = \frac{6.33}{1.392 \times 1.07} \times 1.25 = 5.31$$

   which indicates that a 3/8-in fillet weld is required. The factor 1.25 is a ductility factor from the work of Richard (1986) as modified by Hewitt and Thornton (2004). Even though the stress in this weld is calculated as being uniform, it is well known that there will be local peak stresses, especially in the area where the brace-to-gusset connection comes close to the gusset-to-beam weld. An indication of high stress in this area is also indicated by the Whitmore section cutting into the beam web. Also, as discussed later, frame action will give rise to distortional forces that modify the force distribution given by the UFM.

   c. *Checks on the beam web*: The 627-kip shear is passed into the beam through the gusset-to-beam weld. All of this load is ultimately distributed over the full cross-section of the W14 $\times$ 82, 411 kips passes to the right, and 216 kips are transferred across the column. The length of web required to transmit 627 kips of shear is l_{web}, where $627 = 1.0 \times .6 \times 50 \times .510 \times l_{\text{web}}$. Thus

$$l_{\text{web}} = \frac{627}{1.0 \times 0.6 \times 50 \times 0.51} = 41.0\,\text{in}$$

which is reasonable. Note that this length can be longer than the gusset-to-beam weld, but probably should not exceed about half the beam span.

The vertical component can cause beam web yielding and crippling.

(1) *Web yielding*: The web yield design strength is

$$\phi R_{wy} = 1 \times 0.51 \times 50(51.25 + 2.5 \times 1.45) = 1400 \text{ kips} > 168 \text{ kips, ok}$$

(2) Web crippling: The web crippling design strength is

$$\phi R_{\text{wcp}} = 0.75 \times 0.8 \times t_w^2\left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_y t_f}{t_w}}$$

$$= 0.75 \times 0.8 \times 0.510^2\left[1 + 3\left(\frac{51.25}{14.3}\right)\left(\frac{0.51}{0.855}\right)^{1.5}\right]$$

$$\times \sqrt{\frac{29{,}000 \times 50 \times 0.855}{0.51}} = 1450 \text{ kips} > 168 \text{ kips, ok}$$

The above two checks on the beam web seldom control but should be checked “just in case.” The web crippling formula used is that for locations not near the beam end because the beam-to-column connection will effectively prevent crippling near the beam end. The physical situation is closer to that at some distance from the beam end rather than that at the beam end.

4. *Beam to column*: The loads are 216 kips axial, the specified transfer force and a shear which is the sum of the nominal minimum beam shear of 10 kips and the vertical force from the gusset-to-beam connection of 168 kips. Thus, the total shear is 10 + 168 = 178 kips.

a. Bolts and end plate: As established earlier in this example, the bolt design strength in shear is $\phi r_{\text{str}} = 31.6$ kips. In this connection, since the bolts also see a tensile load, there is an interaction between tension and shear that must be satisfied. If V is the factored shear per bolt, the design tensile strength is

$$\phi r'_t = 1.13\text{T}_\text{b}\left(1 - \frac{V}{\phi r_{\text{str}}}\right) \le 0.75 \times 90A_b$$

This formula is obtained by inverting Specification formula J3-5a. T_b is the bolt pretension of 56 kips for A325 $1^1/_8$-in-diameter bolts and A_b is the bolt nominal area $= \pi/4 \times 1.125^2 = 0.994\ \text{in}^2$.

For $V = 179/10 = 17.9$ kips < 31.6 kips, ok,

$\phi r'_t = 1.13 \times 56(1 - 17.9/31.6) = 27.4$ kips and $0.75 \times 90 \times .994 =$ 67.1 kips.

Thus $\phi r'_t = 27.4$ kips and $\phi R'_t = 10 \times 27.4 = 274$ kips > 216 kips, ok. Checking the interaction for an N-type bearing connection, which also must be done because the serviceability slip-critical condition is used.

$$F'_{nt} = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v \le F_{nt}$$

$$= 1.3 \times 90 - \frac{90}{0.75 \times 48}\left(\frac{17.9}{0.994}\right) = 69.6\,\text{ksi}$$

$$\phi F'_{nt} = 0.75 \times 69.6 = 52.2\,\text{ksi} < 0.75 \times 90 = 67.5\,\text{ksi, ok}$$

$\phi r'_t = 52.2 \times 0.994 = 52.2\,\text{ksi} > 27.4\,\text{kips}$, so bearing does not control.

To determine the end plate thickness required, the critical dimension is the distance "b" from the face of the beam web to the center of the bolts. For $5\frac{1}{2}$-in-cross centers, $b = (5.5 - .5)/2 = 2.5$ in. To make the bolts above and below the flanges approximately equally critical, they should be placed no more than $2\frac{1}{2}$ in above and below the flanges. Figure 2.2 shows them placed at 2 in. Let the end plate be 11 in wide. Then $a = (11 - 5.5)/2 = 2.75 < 1.25 \times 2.5 = 3.125$ ok. The edge distance at the top and bottom of the end plate is 1.5 in, which is more critical than 2.75 in, and will be used in the following calculations. The notation for a and b follows that of the AISC Manual as does the remainder of this procedure.

$$b' = b - \frac{d}{2} = 2.5 - \frac{1.125}{2} = 1.9375$$

$$a' = a + \frac{d}{2} = 1.5 + \frac{1.125}{2} = 2.0625$$

$$\rho = \frac{b'}{a'} = 0.94$$

$$\beta = \frac{1}{\rho}\left(\frac{\phi r'_t}{T} - 1\right)$$

where T = required tension per bolt = 21.6 kips.

$$\beta = \frac{1}{0.94}\left(\frac{27.4}{21.6} - 1\right) = 0.286$$

$$\alpha' = \min\left\{\frac{1}{\delta}\left(\frac{\beta}{1-\beta}\right), 1\right\}$$

where $\delta = 1 - d'/p = 1 - 1.1875/4 = 0.70$

In the above expression, p is the tributary length of end plate per bolt. For the bolts adjacent to the beam web, this is obviously 4 in. For the bolts adjacent to the flanges, it is also approximately

4 in for p since at $b = 2.0$ in, a 45° spread from the center of the bolt gives $p = 4$ in. Note also that p cannot exceed one half of the width of the end plate.

$$\alpha' = \min\left\{\frac{1}{0.70}\left(\frac{0.286}{1 - 0.286}\right), 1\right\} = 0.572$$

The required end plate thickness is

$$t_{req'd}\sqrt{\frac{4.44Tb'}{pF_u(1 + \delta\alpha')}} = \sqrt{\frac{4.44 \times 21.6 \times 1.94}{4.0 \times 58(1 + 0.70 \times 0.572)}} = 0.757 \text{ in}$$

Use a $^7/_8$-in end plate, 11 in wide and $14^1/_4 + 2 + 2 + 1^1/_2 + 1^1/_2 =$ 21.25 in long.

b. Weld of beam to end plate: All of the shear of 179 kips exists in the beam web before it is transferred to the end plate by the weld of the beam to the end plate. The shear capacity of the beam web is

$$\phi R_v = 1.0 \times .6 \times 50 \times .510 \times 14.3 = 219 \text{ kips} > 178 \text{ kips, ok}$$

The weld to the end plate that carries this shear is the weld to the beam web plus the weld around to about the k_1 distance inside the beam profile and $2\,k_1$ on the outside of the flanges. This length is thus

$$2(d - 2t_f) + 4\left(k_1 - \frac{t_w}{2}\right) + 4k_1 = 2 \times (14.31 - 2 \times 0.855)$$
$$+ 4 \times (1 - 0.510/2) + 4 \times 1 = 32.2 \text{ in}$$

The force in this weld per inch due to shear is

$$f_v = \frac{178}{32.2} = 5.53 \text{ kips/in}$$

The length of weld that carries the axial force of 216 kips is the entire profile weld whose length is $4 \times 10.13 - 2 \times 0.51 + 2 \times 14.3 =$ 68.0 in. The force in this weld per inch due to axial force is

$$f_a = \frac{216}{68.0} = 3.18 \text{ kips/in}$$

Also, where the bolts are close together, a "hot spot" stress should be checked. The most critical bolt in this regard is the one at the center of the W14 × 82. The axial force in the weld local to these bolts is

$$f_a' = \frac{2 \times 21.6}{8} = 5.4 \text{ kips/in}$$

The controlling resultant force in the weld is thus

$$f_R = \sqrt{5.53^2 + 5.40^2} = 7.73 \text{ kips/in}$$

To account for the directional strength increase on fillet welds

$$\theta = \tan^{-1}\left(\frac{5.40}{5.53}\right) = 44.3°$$

$$\mu = 1.0 + 0.5 \sin^{1.5}(44.3) = 1.29$$

The required weld size is

$$D = \frac{7.73}{1.392 \times 1.29} = 4.30 \text{ use a 5/16-in fillet weld}$$

As a final check, make sure that the beam web can deliver the axial force to the bolts. The tensile load for 2 bolts is $2 \times 21.6 = 43.2$ kips, and 4 in of the beam web must be capable of delivering this load, that is, providing a load path. The tensile capacity of 4 in of the beam web is $4 \times 0.510 \times 0.9 \times 50 = 91.8$ kips > 43.2 kips, ok.

2.2.1.3 Some observations on the design of gusset plates. It is a tenet of all gusset plate designs that it must be able to be shown that the stresses on any cut section of the gusset do not exceed the yield stresses on this section. Now, once the resultant forces on the gusset horizontal and vertical sections are calculated by the UFM, the resultant forces on any other cut section, such as section a-a of Fig. 2.2, are easy to calculate (see the appropriate free-body diagram incorporating this section, as shown in Fig. 2.4, where the resultant forces on section a-a are shown), but the determination of the stresses is not. The traditional approach

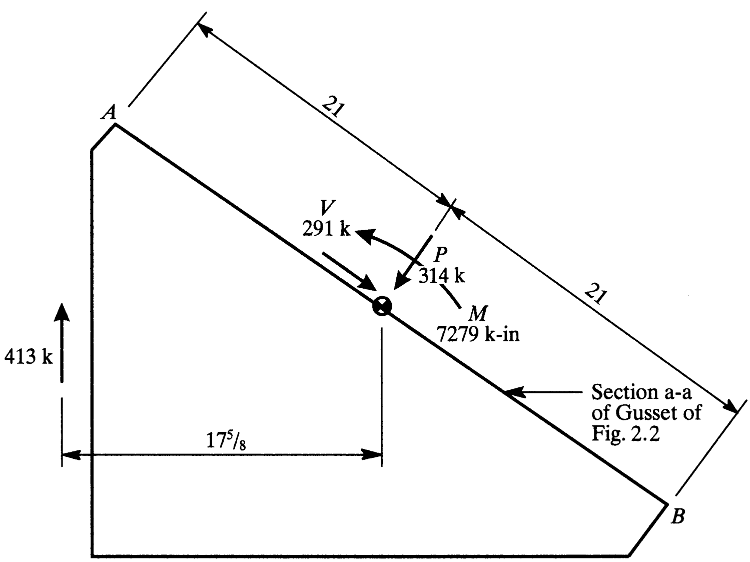


Figure 2.4 Free-body diagram of portion of gusset cut at section a-a of Fig. 2.2.

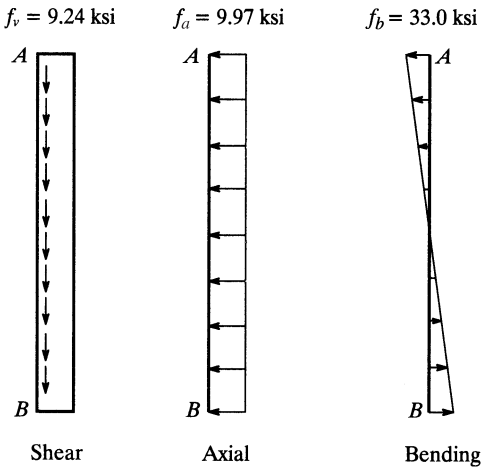


Figure 2.5 Traditional cut section stresses.

to the determination of stresses, as mentioned in many books (Blodgett, 1966; Gaylord and Gaylord, 1972; Kulak et al., 1987) and papers (Whitmore 1952; Vasarhelyi, 1971), is to use the formulas intended for long slender members, that is $f_a = P/A$ for axial stress, $f_b = Mc/I$ for bending stress, and $f_v = V/A$ for shear stress. It is well known that these are not correct for gusset plates (Timoshenko, 1970). They are recommended only because there is seemingly no alternative. Actually, the UFM, coupled with the Whitmore section and the block shear fracture limit state, is an alternative as will be shown subsequently.

Applying the slender member formulas to the section and forces of Fig. 2.4, the stresses and stress distribution of Fig. 2.5 result. The stresses are calculated as

$$\text{shear: } f_v = \frac{291}{0.75 \times 42} = 9.24 \text{ ksi}$$

$$\text{axial:} f_a = \frac{314}{0.75 \ \times \ 42^2} = 9.97 \text{ ksi}$$

$$\text{bending: } f_b = \frac{7280 \times 6}{0.75 \times 42^2} = 33.0 \text{ ksi}$$

These are the basic "elastic"* stress distributions. The peak stress occurs at point A and is

$$\text{shear: } f_v = 9.24 \text{ ksi}$$

$$\text{normal: } f_a + f_b = 9.97 + 33.0 = 43.0 \text{ ksi}$$

*Actually the shear stress is not elastic because it is assumed uniform. The slender beam theory elastic shear stress would have a parabolic distribution with a peak stress of $9.24 \times 1.5 = 13.9$ ksi at the center of the section.

The shear yield stress (design strength) is $\phi F_v = \phi(0.6\,F_y) = 1.0(0.6 \times 36) = 21.6$ ksi. Since $9.24 < 21.6$, the section has not yielded in shear. The normal yield stress (design strength) is $\phi F_n = \phi F_y = 0.9\,(36) = 32.4$ ksi. Since $43.0 > 32.4$, the yield strength has been exceeded at point A. At this point, it appears that the design is unsatisfactory (i.e., not meeting AISC requirements). But consider that the normal stress exceeds yield over only about 11 inches of the 42 in-long section starting from point A. The remaining $42 - 11 = 31$ in., have not yet yielded. This means that failure has not occurred because the elastic portion of the section will constrain unbounded yield deformations, that is, the deformation is "self-limited." Also, the stress of 43.0 ksi is totally artificial! It cannot be achieved in an elastic–perfectly plastic material with a design yield point of 32.4 ksi. What *will* happen is that when the design yield point of 32.4 ksi is reached, the stresses on the section will redistribute until the design yield point is reached at *every* point of the cross section. At this time, the plate will fail by unrestrained yielding if the applied loads are such that higher stresses are required for equilibrium.

To conclude on the basis of 43.0 ksi at point A, that the plate has failed is thus false. What must be done is to see if a redistributed stress state on the section can be achieved which nowhere exceeds the design yield stress. Note that if this can be achieved, all AISC requirements will have been satisfied. The AISC specifies that the design yield stress shall not be exceeded, but does *not* specify the formulas used to determine this.

The shear stress f_v and the axial stress f_a are already assumed uniform. Only the bending stress f_b is nonuniform. To achieve simultaneous yield over the entire section, the bending stress must be adjusted so that when combined with the axial stress, a uniform normal stress is achieved. To this end, consider Fig. 2.6. Here the bending stress is assumed uniform but of different magnitudes over the upper and lower parts of the

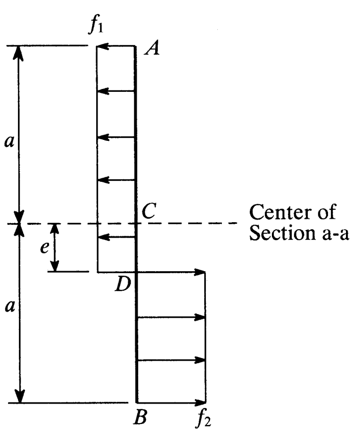


Figure 2.6 Admissible bending stress distribution of section a-a.

section. Note that this can be done because **M** of Fig. 2.4, although shown at the centroid of the section, is actually a free vector that can be applied anywhere on the section or indeed anywhere on the free-body diagram. This being the case, there is no reason to assume that the bending stress distribution is symmetrical about the center of the section. Considering the distribution shown in Fig. 2.6, because the stress from A to the center is too high, the zero point of the distribution can be allowed to move down the amount e toward B. Equating the couple **M** of Fig. 2.4 to the statically equivalent stress distribution of Fig. 2.6 and taking moments about point D,

$$\mathbf{M} = \frac{t}{2}[f_1(a + e)^2 + f_2(a - e)^2]$$

where t is the gusset thickness. Also, from equilibrium

$$f_1\,(a + e)\,t = f_2\,(a - e)t$$

The above two equations permit a solution for f_1 and f_2 as

$$f_1 = \frac{\mathbf{M}}{at(a + e)}$$

$$f_2 = \frac{\mathbf{M}}{at(a - e)}$$

For a uniform distribution of normal stress,

$$f_1 + f_a = f_2 - f_a$$

from which e can be obtained as

$$e = \frac{1}{2}\left[\sqrt{\left(\frac{\mathbf{M}}{at\,f_a}\right)^2 + 4a^2} - \frac{\mathbf{M}}{at\,f_a}\right]$$

Substituting numerical values,

$$e = \frac{1}{2}\left[\sqrt{\left(\frac{7280}{(21)(0.75)(9.97)}\right)^2 + 4(21)^2} - \frac{7280}{(21)(0.75)(9.97)}\right] = 8.10 \text{ in}$$

Thus,

$$f_1 = \frac{7280}{(21)(0.75)(21 + 8.10)} = 15.9 \text{ ksi}$$

$$f_2 = \frac{7280}{(21)(0.75)(21 - 8.10)} = 35.8 \text{ ksi}$$

and the normal stress at point A is

$$f_{n_A} = f_1 + f_a = 15.9 + 9.97 = 25.9 \text{ ksi}$$

and at point B

$$f_{n_B} = f_2 - f_a = 35.8 - 9.97 = 25.9 \text{ ksi}$$

Now the entire section is uniformly stressed. Since

$$f_v = 9.24 \text{ ksi} < 21.6 \text{ ksi}$$

$$f_n = 25.9 \text{ ksi} < 32.4 \text{ ksi}$$

at all points of the section, the design yield stress is nowhere exceeded and the connection is satisfactory.

It was stated previously that there is an alternative to the use of the inappropriate slender beam formulas for the analysis and design of gusset plates. The preceding analysis of the special section a-a demonstrates the alternative that results in a true limit state (failure mode or mechanism) rather than the fictitious calculation of "hot spot" point stresses, which since their associated deformation is totally limited by the remaining elastic portions of the section, cannot correspond to a true failure mode or limit state. The UFM performs exactly the same analysis on the gusset horizontal and vertical edges, and on the associated beam-to-column connection. It is capable of producing forces on all interfaces that give rise to uniform stresses. Each interface is designed to just fail under these uniform stresses. Therefore, true limit states are achieved at every interface. For this reason, the UFM achieves a good approximation to the greatest lower bound solution (closest to the true collapse solution) in accordance with the lower bound theorem of limit analysis.

The UFM is a complete departure from the so-called traditional approach to gusset analysis using slender beam theory formulas. It has been validated against all known full-scale gusseted bracing connection tests (Thornton, 1991, 1995b). It does not require the checking of gusset sections such as that studied in this section (section a-a of Fig. 2.4). The analysis at this section was done to prove a point. But the UFM does include a check in the brace-to-gusset part of the calculation that is closely related to the special section a-a of Fig. 2.4. This is the block shear rupture of Fig. 2.7 (Hardash and Bjorhovde, 1985, and Richard, 1983), which is included in section J4 of the AISC Specification (AISC, 2005). The block shear capacity was previously calculated as 877 kips.

Comparing the block shear limit state to the special section a-a limit state, a reserve capacity in block shear $= \dfrac{877 - 855}{855}100 = 2.57\,\%$

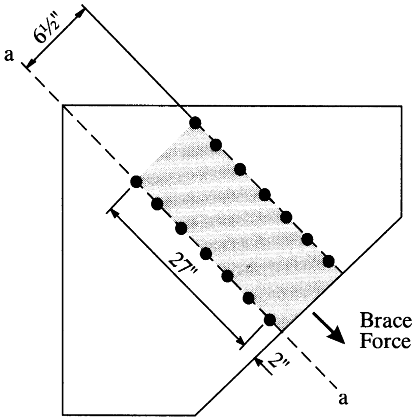


Figure 2.7 Block shear rupture and its relation to gussed section a-a.

is found, and the reserve capacity of the special section = $\frac{32.4 - 25.9}{25.9}100 = 25.1\%$, which shows that block shear gives a conservative prediction of the capacity of the closely related special section.

A second check on the gusset performed as part of the UFM is the Whitmore section check. From the Whitmore section check performed earlier, the Whitmore area is

$$A_w = (37.7 - 10.4) \times 0.75 + 10.4 \times 0.510 \times \frac{50}{36} = 27.8\,\text{in}^2$$

and the Whitmore section design strength in tension is

$$\phi F_w = \phi(F_y \times A_w) = 0.9(36 \times 27.8) = 901\,\text{kips}$$

The reserve capacity of the Whitmore section in tension is $\frac{901 - 855}{855} \times 100 = 5.38\%$, which again gives a conservative prediction of capacity when compared to the special section a-a.

With these two limit states, block shear rupture and Whitmore, the special section limit state is closely bounded and rendered unnecessary. The routine calculations associated with block shear and Whitmore are sufficient in practice to eliminate the consideration of any sections other than the gusset-to-column and gusset-to-beam sections.

2.2.1.4 Example 2. Example bracing connection. This connection is shown in Fig. 2.8. The member on the right of the joint is a "collector" that adds load to the bracing truss. The brace consists of two MC12 $\times$ 45s with toes $1\frac{1}{2}$ in apart. The gusset thickness is thus chosen to be $1\frac{1}{2}$ in and is then checked. The completed design is shown in Fig. 2.8. In this case,

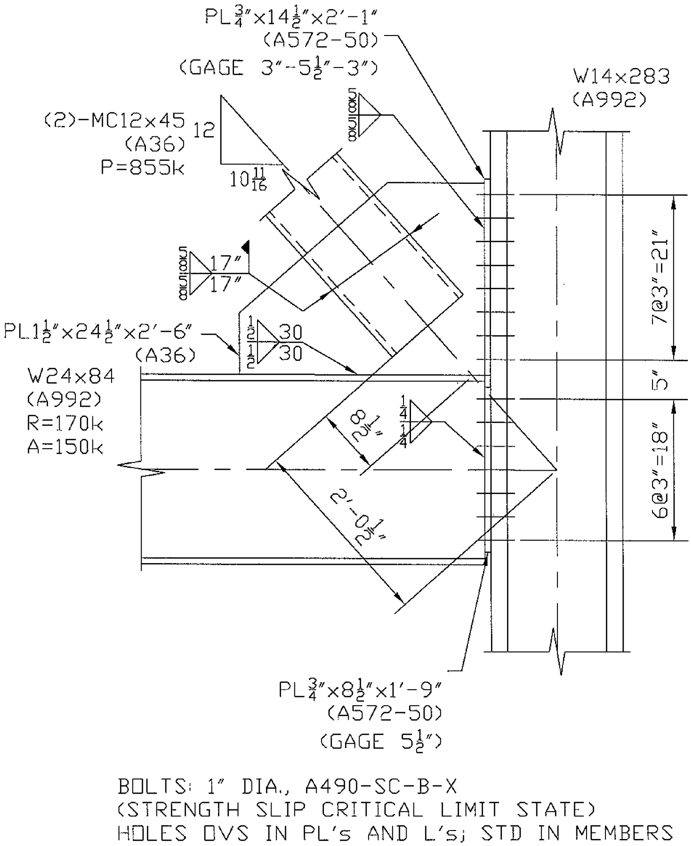


Figure 2.8 Example 2, bracing connection design.

because of the high specified beam shear of 170 kips, it is proposed to use a special case of the UFM which sets the vertical component of the load between the gusset and the beam, V_B, to zero. Figure 2.9 shows the resultant force distribution. This method is called "special case 2" of the UFM and is discussed in the AISC books (AISC, 1992, 1994).

1. *Brace-to-gusset connection:*

   a. *Weld:* The brace is field welded to the gusset with fillet welds. Because of architectural constraints, the gusset size is to be kept to 31 in horizontally and 24½ in vertically. From the geometry of the gusset and brace, about 17 in of fillet weld can be accommodated. The weld size is

$$D = \frac{855}{4 \times 17 \times 1.392} = 9.03$$

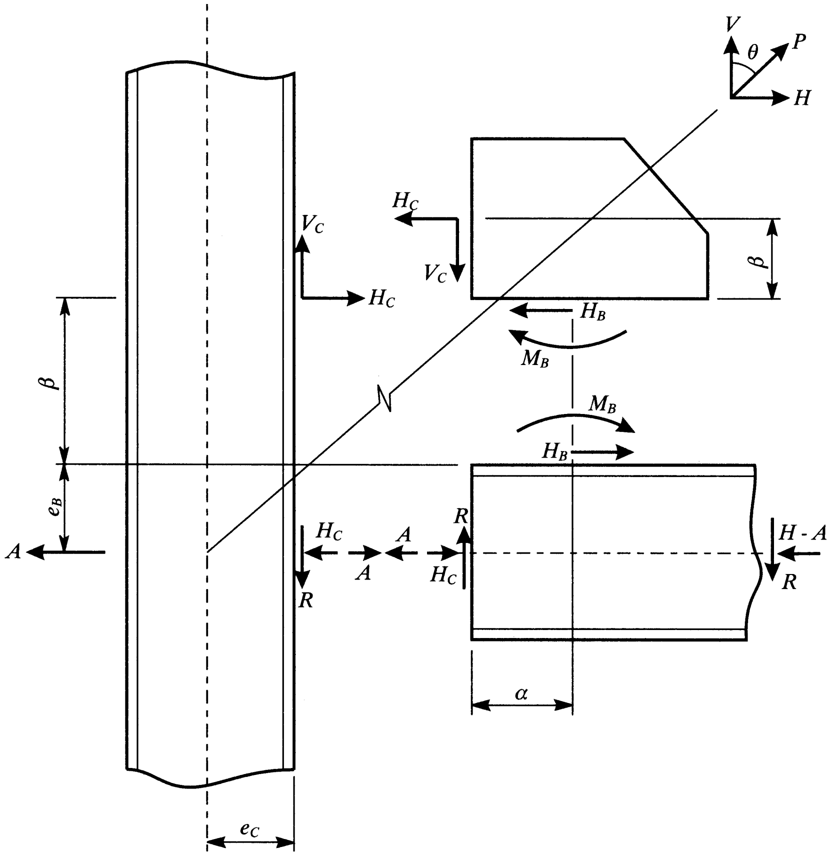


Figure 2.9 Force distribution for special case 2 of the uniform force method.

A $^5/_8$-in fillet weld is indicated, but the flange of the MC12 × 45 must be checked to see if an adequate load path exists. The average thickness of 0.700 in occurs at the center of the flange, which is 4.012 in wide. The thickness at the toe of the flange, because of the usual inside flange slope of $^2/_{12}$ or $16^2/_3\%$, is $0.700 - {^2/_{12}} \times 2.006 = 0.366$ in (see Fig. 2.10). The thickness at the toe of the fillet is $0.366 + 2/12 \times 0.625 = 0.470$ in. The design shear rupture strength of the MC12 flange at the toe of the fillet is

$$\phi R_v = 0.75 \times 0.6 \times 58 \times 0.470 \times 17 \times 4 = 834 \text{ kips}$$

The design tensile rupture strength of the toe of the MC flange under the fillet is

$$\phi R_t = 0.75 \times 36\left(\frac{0.366 + 0.470}{2}\right)0.625 \times 4 = 28 \text{ kips}$$

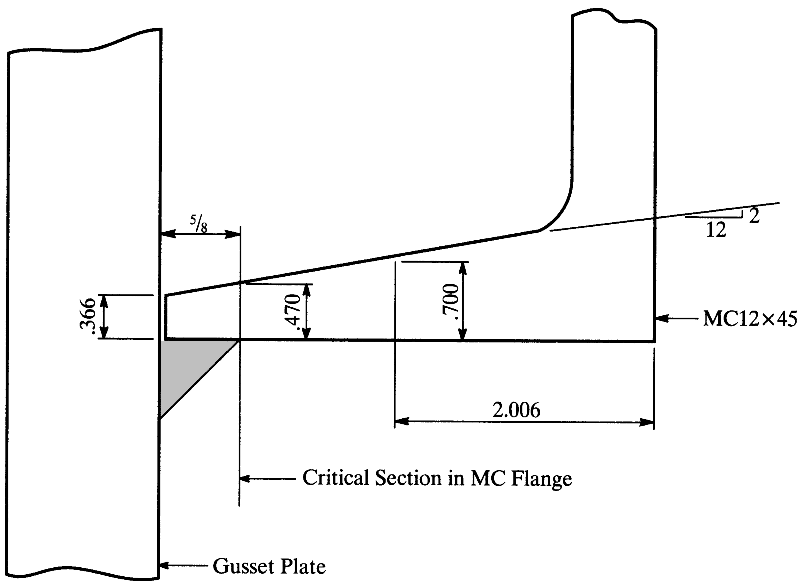


Figure 2.10 Critical section at toe of fillet weld.

Thus the total strength of the load path in the channel flange is 834 + 28 = 862 kips > 855 kips, ok.

b. Gusset-to-brace block shear:
shear yeilding:

$$\phi R_v = 0.90 \times 0.6 \times 36 \times 1.5 \times 17 \times 2 = 991 \text{ kips}$$

tension fracture

$$\phi R_t = 0.75 \times 58 \times 1.5 \times 12 = 783 \text{ kips}$$
$$\phi R_{bs} = 991 + 783 = 1770 \text{ kips} > 855 \text{ kips, ok}$$

c. Whitmore section: The theoretical length of the Whitmore section is (17 tan 30)2 + 12 = 31.6 in. The Whitmore section extends into the column by 5.40 in. The column web is stronger than the gusset since $1.29 \times {}^{50}/_{36} = 1.79 > 1.5$ in. The Whitmore also extends into the beam web by 6.80 in, but since $0.470 \times {}^{50}/_{36} = 0.653 < 1.5$ in, the beam web is not as strong as the gusset. The effective Whitmore section length is

$$l_{\text{weff}} = (31.6 - 6.80) + 6.80 \times \frac{0.470}{1.5} \times \frac{50}{36} = 27.8 \text{ in}$$

The effective length is based on $F_y = 36$ and the gusset thickness of 1.5 in.

Since the brace force can be tension or compression, compression will control. The slenderness ratio of the unsupported length of gusset is

$$\frac{Kl}{r} = \frac{0.5 \times 8.5\sqrt{12}}{1.5} = 9.82$$

The use of K = 0.5 comes from the work of Gross (1990).

Since $Kl/r < 25$

$$\phi F_a = 0.9F_y = 0.9 \times 36 = 32.4 \text{ ksi}$$

and the buckling strength of the gusset is

$$\phi R_{wb} = 27.8 \times 1.5 \times 32.4 = 1350 > 855 \text{ kips, ok}$$

This completes the brace-to-gusset part of the design. Before proceeding, the distribution of forces to the gusset edges must be determined. From Fig. 2.8,

$$e_B = \frac{24.10}{2} = 12.05 \quad e_C = 8.37 \quad \bar{\beta} = 12.5 \quad \bar{\alpha} = 15.0$$

$$\theta = \tan^{-1}\left(\frac{10.6875}{12}\right) = 41.6°$$

$$V_C = P\cos\theta = 855 \times 0.747 = 638 \text{ kips}$$

$$H_C = \frac{V_C e_C}{e_B + \beta} = \frac{638 \times 8.37}{12.05 + 12.5} = 218 \text{ kips}$$

$$H_B = P\sin\theta - H_C = 855 \times 0.665 - 218 = 351 \text{ kips}$$

$$M_B = H_B e_B = 351 \times 12.05 = 4230 \text{ kips-in}$$

Note that, in this special case 2, the calculations can be simplified as shown here. The same results can be obtained formally with the UFM by setting $\beta = \bar{\beta} = 12.5$ and proceeding as follows. With $\tan\theta = 0.8906$,

$$\alpha - 0.8906\beta = 12.05 \times 0.8906 - 8.37 = 2.362$$

Setting $\beta = \bar{\beta} = 12.5$, $\alpha = 13.5$. Since $\bar{\alpha}$ is approximately 15.0, there will be a couple, M_B, on the gusset-to-beam edge. Continuing

$$r = \sqrt{(13.5 + 8.37)^2 + (12.5 + 12.05)^2} = 32.9$$

$$\frac{P}{r} = \frac{855}{32.9} = 26.0$$

$$H_B = \frac{\alpha}{r}P = 351 \text{ kips}$$

$$H_C = \frac{e_C}{r}P = 218 \text{ kips}$$

$$V_B = \frac{e_B}{r}P = 313 \text{ kips}$$

$$V_C = \frac{\beta}{r}P = 325 \text{ kips}$$

$$M_B = |V_B(\alpha - \overline{\alpha})| = 470 \text{ kips-in}$$

This couple is clockwise on the gusset edge. Now, introducing special case 2, in the notation of the AISC *Manual of Steel Construction* (2005), set $\Delta V_B = V_B = 313$ kips. This reduces the vertical force between the gusset and beam to zero, and increases the gusset-to-column shear, V_C, to 313 + 325 = 638 kips and creates a counterclockwise couple on the gusset-to-beam edge of $\Delta V_B\overline{\alpha} = 313 \times 15.0 = 4700$ kips-in. The total couple on the gusset-to-beam edge is thus $M_B = 4700 - 470 = 4230$ kips-in. It can be seen that these gusset interface forces are the same as those obtained from the simpler method.

2. *Gusset-to-column connection*: The loads are 638 kips shear and 218 kips axial.

   a. *Gusset stresses*:

$$f_v = \frac{638}{1.5 \times 24.5} = 17.4 \text{ ksi} < 1.0 \times 0.6 \times 36 = 21.6 \text{ ksi, ok}$$

$$f_a = \frac{2/8}{1.5 \times 24.5} = 5.93 \text{ ksi} < 0.9 \times 36 = 32.4 \text{ ksi, ok}$$

   b. *Weld of gusset to end plate*: Using AISC LRFD, Table 8-4, $P_u = \sqrt{638^2 + 218^2} = 674$ kips and the angle from the longitudinal weld axis is $\tan^{-1}(218/638) = 18.9°$, so using the table for 15° with k = a = 0.0, c = 3.84. Thus,

$$D = \frac{674}{0.75 \times 3.84 \times 24.5} = 9.55$$

   which indicates that a 5/8 fillet is required. No ductility factor is used because the flexibility of the end plate will enable redistribution of nonuniform weld stresses.

   (1) *Check bolt capacity*
   The bolts are A490 SC-B-X in OVS holes. The slip-critical strength criterion is used because slip into bearing in this building could cause excessive P-Δ effects. Thus, from Table 7-4

$$\phi r_v = 18.3 \times 1.43 = 26.2 \text{ kips/bolt}$$

and from Table 7-2

$$\phi r_t = 66.6 \text{ kips/bolt}$$

(2) *Bolt shear*

$$\phi R_v = 26.2 \times 8 \times 4 = 838 \text{ kips} > 638 \text{ kips, ok}$$

(3) *Bolt tension*
Since only the two inside columns of bolts are effective in carrying the tension,

$$\phi R_t = 66.6 \times 8 \times 2 = 1070 \text{ kips} > 218 \text{ kips, ok}$$

(4) *Bolt shear / tension interaction*
Because the slip-critical strength criterion is used, no bearing checks are required for these bolts. The interaction equation for slip-critical bolts is given in Specification Section J3.9 as,

$$\phi r_v' = \phi r_v\left(1 - \frac{T_u}{D_u T_b}\right)$$

where $\phi r_v'$ = reduced shear strength
T_u = tension load per bolt
D_u = 1.13
T_b = specified bolt pretension, 64 kips for 1-in A490 bolts

therefore,

$$\phi r_v' = 26.2\left(1 - \frac{218/16}{1.13 \times 64}\right) = 21.3 \text{ kips/bolt} > \frac{638}{32}$$
$$= 20.0 \text{ kips/bolt, ok}$$

(5) *End plate thickness required and prying action*
The prying action formulation of the Manual, pages 9-10 through 9-13, requires that the bolt tension strength $\phi r_t'$ be determined as a function of the shear load per bolt. This requires that the interaction equation of Specification Section J3.9 be inverted, which can be shown to give

$$\phi r_t' = D_u T_b\left(1 - \frac{V_u}{\phi r_v}\right)$$

where $\phi r_v'$ = reduced tensile strength
V_u = shear load per bolt
D_u and T_b are as previously defined.

Therefore

$$\phi r_t' = 1.13 \times 64\left(1 - \frac{20.0}{26.2}\right) = 17.1 \text{ kips/bolt} >$$
$$13.6 \text{ kips/bolt, ok}$$

Try a 1-in-thick end plate of A572-Grade 50 steel. Following the notation of the Manual.

$$b = \frac{5.5 - 1.5}{2} = 2 \text{ in} \qquad a = \frac{14.5 - 5.5}{2} = 4.5 \text{ in}$$

Check $a \leq 1.25b = 1.25 \times 2.00 = 2.50$. Therefore, use $a = 2.50$ in.

In this problem, "a" should not be taken as larger than the bolt gage of 3 in.

$$b' = 2.00 - \frac{1.00}{2} = 1.5 \text{ in} \qquad a' = 2.50 + \frac{1.00}{2} = 3.00 \text{ in}$$

$$\rho = \frac{b'}{a'} = \frac{1.50}{3.00} = 0.50$$

$$\delta = 1 - \frac{d'}{p} = 1 - \frac{1.25}{3.00} = 0.583$$

$$B = \phi r_t' = 17.1$$

$$t_c = \sqrt{\frac{4.44Bb'}{pF_u}} = \sqrt{\frac{4.44 \times 17.1 \times 1.5}{3 \times 65}} = 0.764 \text{ in}$$

$$\alpha' = \frac{1}{\delta(1+p)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right] = \frac{1}{0.583 \times 1.50}\left[\left(\frac{0.764}{1.0}\right)^2 - 1\right]$$

$$= -0.474$$

Use $\alpha' = 0$ and $Q = 1.00$

$$T_{\text{avail}} = QB = 1.00 \times 17.1 = 17.1 \text{ kips/bolt}$$
$$> 13.6 \text{ kips/bolt} \therefore \text{ bolts okay}$$

Since $t_c = 0.764$ in $< t = 1.00$ in it is possible that a thinner end plate can be used. Try $t = 3/4$ in.

$$\alpha' = \frac{1}{\delta(1+p)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right] = \frac{1}{0.583 \times 1.50}\left[\left(\frac{0.764}{0.75}\right)^2 - 1\right] = 0.043$$

$$Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta\alpha') = \left(\frac{0.75}{0.764}\right)^2 (1 + 0.583 \times 0.043) = 0.988$$

$$T_{\text{avail}} = QB = 0.988 \times 17.1 = 16.9 \text{ kips/bolt}$$
$$> 13.6 \text{ kips/bolt} \therefore \text{ bolts}$$

Use $3/4$-in end plate.
Check clearance
From Table 7-16

$$C_3 = 1 \text{ in} < \frac{5.5 - 1.5}{2} - 0.625 = 1.375 \text{ in} \therefore \text{ ok}$$

(6) *Check column flange prying*
Since $t_f = 2.07$ in and $t_w = 1.29$ in., it is obvious that this limit state will not govern. In addition to the prying check, the end plate should be checked for gross shear, net shear, and block shear. These will not govern in this case.

c. *Checks on column web:*

(1) *Web yielding (under normal load H_c)*:

$$\phi R_{wy} = 1.0 \times 50 \times 1.290\left(24.5 + 5 \times 2\frac{3}{4}\right)$$
$$= 2470 \text{ kips} > 218 \text{ kips, ok}$$

(2) *Web crippling (under normal load H_c)*:

$$\phi R_{wcp} = 0.75 \times 0.8 \times 1.29^2\left[1 + 3\left(\frac{24.5}{16.7}\right)\left(\frac{1.29}{2.07}\right)^{1.5}\right]$$
$$\times \sqrt{\frac{29000 \times 50 \times 2.07}{1.29}} = 4820 \text{ kips} > 218 \text{ kips, ok}$$

(3) *Web shear:* The horizontal force, H_c, is transferred to the column by the gusset-to-column connection and back into the beam by the beam-to-column connection. Thus, the column web sees $H_e = 218$ kips as a shear. The column shear capacity is

$$\phi R_v = 1.0 \times 0.6 \times 50 \times 1.29 \times 16.7 = 646 \text{ kips} > 218 \text{ kips, ok}$$

3. *Gusset-to-beam connection*: The loads are 351 kips shear and a 4230-kips-in couple.

a. *Gusset stresses*:

$$f_v = \frac{351}{1.5 \times 30} = 7.80 \text{ ksi} < 21.6 \text{ ksi, ok}$$
$$f_b = \frac{4230 \times 4}{1.5 \times 30^2} = 12.5 \text{ ksi} < 32.4 \text{ ksi, ok}$$

b. *Weld of gusset-to-beam flange*:

$$f_{\text{peak}} = \sqrt{7.80^2 + 12.5^2} \times \frac{1.5}{2} = 11.0 \text{ kips/in}$$
$$\theta_w = \tan^{-1}\left(\frac{12.5}{7.80}\right) = 58.0°$$
$$f_{\text{ave}} = \frac{1}{2}\left[\sqrt{7.80^2 + 12.5^2} + \sqrt{7.80^2 + 12.5^2}\,\right]\frac{1.5}{2} = 11.0 \text{ kips/in}$$

Since 11.0/11.0 = 1.0 < 1.25, the weld size based on the average force in the weld, $f_{ave} \times 1.25$, therefore

$$D = \frac{11.0 \times 1.25}{1.392(1 + 0.5\sin^{1.5}(58.2))} = 7.09$$

A $\frac{1}{2}$ fillet weld is indicated. The 1.25 is the ductility factor; see Hewitt and Thornton (2004).

An alternate method for calculating the weld size required is to use Table 8-38 of the AISC *Manual of Steel Construction* (2005), special case $k = 0$, $P_u = 349$, and $al = 4205/349 = 12.05$ in; thus $a = 12.05/30 = 0.40$ and $c = 2.00$, and the required weld size is

$$\mathrm{D} = \frac{349}{2.0 \times 30} = 5.8$$

A $\frac{3}{8}$ fillet is indicated. This method does not give an indication of peak and average stresses, but it will be safe to use the ductility factor. Thus, the required weld size would be

$$\mathrm{D} = 5.8 \times 1.25 = 7.25$$

Thus, by either method, a 1/2 fillet is indicated.

c. *Checks on beam web:*

(1) *Web yield:* Although there is no axial component, the couple M_B = 4230 kips-in is statically equivalent to equal and opposite vertical shears at a lever arm of one-half the gusset length or 15 in. The shear is thus

$$V_s = \frac{4230}{15} = 282 \text{ kips}$$

This shear is applied to the flange as a transverse load over 15 in of flange. It is convenient for analysis purposes to imagine this load doubled and applied over the contact length N = 30 in. The design web yielding strength is

$$\begin{aligned}\phi R_{wy} &= 1.0 \times 50 \times 0.47(30 + 2.5 \times 1.27) = 780 \text{ kips} > 282 \times 2 \\ &= 564 \text{ kips, ok}\end{aligned}$$

(2) *Web crippling:*

$$\phi R_{\mathrm{wcp}} = 0.75 \times 0.8 \times 0.47^2\left[1 + 3\left(\frac{30}{24.1}\right)\left(\frac{0.47}{0.77}\right)^{1.5}\right]$$

$$\times\sqrt{\frac{29000 \times 50 \times 0.77}{0.47}} = 568 \text{ kips} > 564 \text{ kips, ok}$$

(3) *Web shear:*

$$\phi P_v = 1.0 \times 0.6 \times 50 \times 0.47 \times 24.1 = 340 \text{ kips} > 282 \text{ kips, ok}$$

The maximum shear due to the couple is centered on the gusset 15 in from the beam end. It does not reach the beam-to-column connection where the beam shear is 170 kips. Because of the total vertical shear capacity of the beam and the gusset acting together, there is no need to check the beam web for a combined shear of V_s and R of 282 + 170 = 452 kips.

4. *Beam-to-column connection*: The shear load is 170 kips and the axial force is H_c +/− A = 218 +/− 150 kips. Since the W18 × 50 is a collector, it adds load to the bracing system. Thus, the axial load is 218 + 150 = 368 kips. However, the AISC book on connections (AISC, 1992) addresses this situation and states that because of frame action (distortion), which will always tend to reduce H_C, it is reasonable to use the larger of Hc and A as the axial force. Thus the axial load would be 218 kips in this case. It should be noted however that when the brace load is not due to primarily lateral loads frame action might not occur.

   a. Bolts and end plate: Though loads caused by wind and seismic forces are not considered cyclic (fatigue) loads and bolts in tension are not required to be designed as slip critical, the bolts are specified to be designed as A490 SC-B-X 1-in diameter to accommodate the use of oversize 1¼-in-diameter holes. As mentioned earlier the slip-critical strength criterion in used. Thus, for shear

$$\phi r_v = 26.2 \text{ kips/bolt}$$

and for tension

$$\phi r_t = 66.6 \text{ kips/bolt}$$

The end plate is ¾ in thick with seven rows and 2 columns of bolts. Note that the end plate is 14½ in wide for the gusset to column connection and 8½ in wide for the beam-to-column connection.

For shear

$$\phi R_v = 26.2 \times 14 = 367 \text{ kips} > 170 \text{ kips, ok}$$

For tenion

$$\phi R_t = 66.6 \times 14 = 932 \text{ kips} > 218 \text{ kips, ok}$$

Bolt bearing and tearout need not be checked because the slip critical strength limit state has been used.

For tension, the bolts and end plate are checked together for prying action.

Since all of the bolts are subjected to tension simultaneously, there is interaction between tension and shear. The reduced tensile capacity is

$$B = \phi r_t' = 1.13 \times 64 \left(1 - \frac{170/14}{20.3}\right) = 29.1 \text{ kips/bolt}$$

Since 29.1 kips > 218/14 = 15.7 kips, the bolts are ok for tension. The bearing type interaction expression would also be checked if the serviceability slip-critical limit state were used. Prying action is now checked using the method and notation of the AISC *Manual of Steel Construction* (2005), pages 9-10 through 9-13:

$$b = \frac{5.5 - 0.47}{2} = 2.52$$

$$a = \frac{8.5 - 5.5}{2} = 1.50$$

Check $1.25b = 1.25 \times 2.52 = 3.15$. Since $3.15 > 1.50$, use $a = 1.50$.

$$b' = 2.52 - \frac{1.0}{2} = 2.02$$

$$a' = 1.50 + \frac{1.0}{2} = 2.00$$

$$\rho = 1.01$$

$$\delta = 1 - \frac{1.25}{3} = 0.583$$

$$t_c = \sqrt{\frac{4.44 \times 29.1 \times 2.02}{3 \times 58}} = 1.22$$

$$\alpha' = \frac{1}{0.583 \times 2.01}\left[\left(\frac{1.22}{0.75}\right)^2 - 1\right] = 1.40$$

Use $\alpha' = 1.00$

The design strength per bolt including prying is

$$\phi T = 29.1\left(\frac{0.75}{1.22}\right)^2 (1 + 0.583 \times 1.00) = 17.4 \text{ kips} > 17.4 \text{ kips, ok}$$

In addition to the prying check, the end plate should also be checked for gross shear net shear and block shear. These will not control in this case.

b. Weld of end plate to beam web: The weld is a double line weld with length $l = 21$ in, $k = a = 0$. From the AISC *Manual of Steel*

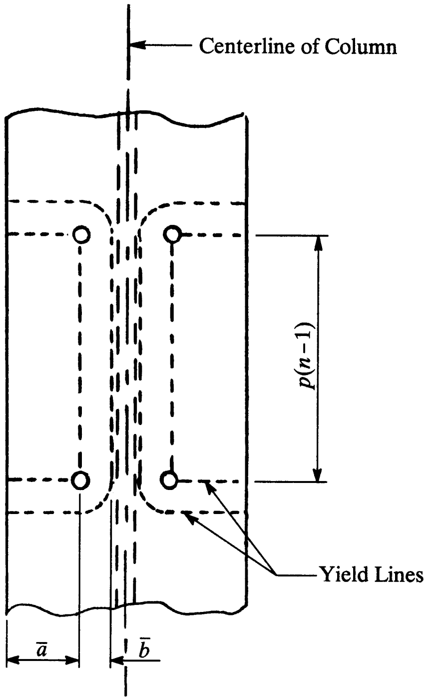


Figure 2.11 Yield lines for flange bending.

Construction (2005), Table 8-4. Since $\tan^{-1} 220/170 = 52.3°$, use the chart for 45°. With $C = 4.64$ a $^{1}/_{4}$ fillet weld has a capacity of $\phi R_w = 0.75 \times 4.64 \times 4 \times 21 = 292$ kips. Thus, since 292 kips > $\sqrt{218^2 + 170^2} = 278$ kips, the 1/4 fillet weld is ok. The web thickness required to support this weld is

$$t_{\min} = \frac{6.19 \times 4}{65}\left(\frac{278}{292}\right) = 0.363 \text{ in} < 0.478 \text{ in, ok}$$

c. *Bending of the column flange*: As was the case for the gusset to column connection, since $t_f = 2.07$ in is much greater than the end plate thickness of $^{3}/_{4}$ in, the check can be ignored. The following method can be used when t_f and t_p are of similar thicknesses. Because of the axial force, the column flange can bend just as the clip angles. A yield-line analysis derived from Mann and Morris (1979) can be used to determine an effective tributary length of column flange per bolt. The yield lines are shown in Fig. 2.11. From Fig. 2.11,

$$p_{\text{eff}} = \frac{(n - 1)p + \pi\bar{b} + 2\bar{a}}{n}$$

where $\bar{b} = (5.5 - 1.29)/2 = 2.11$
$\bar{a} = (16.1 - 5.5)/2 = 5.31$
$p = 3$
$n = 7$

Thus,

$$p_{\text{eff}} = \frac{6 \times 3 + \pi \times 2.11 + 2 \times 5.31}{7} = 5.03$$

Using p_{eff} in place of p, and following the AISC procedure,

$$b = \bar{b} = 2.11$$

$$b' = 2.11 - \frac{1.0}{2} = 1.61$$

$$a = \min\left(\frac{4 + 4 + 0.47 - 5.5}{2}, 5.31, 1.25 \times 2.11\right)$$

$$= \min(1.48, 5.31, 2.63) = 1.49$$

$$a' = 1.49 + 0.5 = 1.99$$

$$p = \frac{b'}{a'} = 0.81$$

$$\delta = 1 - \frac{1.06}{5.03} = 0.79$$

Note that standard holes are used in the column flange.

$$t_c = \sqrt{\frac{4.44 \times 29.1 \times 1.61}{5.03 \times 65}} = 0.798$$

$$\alpha' = \frac{1}{0.79 \times 1.81}\left[\left(\frac{0.798}{2.07}\right)^2 - 1\right] = -0.595$$

Since $\alpha' < 0$, use $\alpha' = 0$

$$\phi T = 29.1 \text{ kips/bolt} > 15.7 \text{ kips/bolt, ok}$$

When $\alpha' < 1$, the bolts, and not the flange, control the strength of the connection.

2.2.1.5 Frame action. The method of bracing connection design presented here, the uniform force method (UFM), is an equilibrium-based method. Every proper method of design for bracing connections, and in fact for every type of connection, must satisfy equilibrium. The set of forces derived from the UFM, as shown in Fig. 2.3, satisfy equilibrium of the gusset, the column, and the beam with axial forces only. Such a set of forces is said to be "admissible." But equilibrium is not the only requirement that must be satisfied to establish the true distribution of forces in a structure or connection. Two additional requirements are the constitutive equations that relate forces to deformations and the compatibility equations that relate deformations to displacements.

If it is assumed that the structure and connection behave elastically (an assumption as to constitutive equations) and that the beam and the column remain perpendicular to each other (an assumption as to deformation–displacement equations), then an estimate of the moment in the beam due to distortion of the frame (frame action) (Thornton, 1991) is given by

$$M_D = 6\frac{P}{Abc}\,\frac{I_b I_c}{\left(\dfrac{I_b}{b} + \dfrac{2I_c}{c}\right)}\,\frac{(b^2 + c^2)}{bc}$$

where D = distortion
I_b = moment of inertia of beam = 2370 in^4
I_c = moment of inertia of column = 3840 in^4
P = brace force = 855 kips
A = brace area = 26.4 in^2
b = length of beam to inflection point (assumed at beam midpoint) = 175 in
c = length of columns to inflection points (assumed at column midlengths) = 96 in

With $\dfrac{2I_c}{c} = 80$ and $\dfrac{I_b}{b} = 13.5$

$$M_D = \frac{6 \times 855 \times 2370 \times 3840}{26.4 \times 175 \times 96 \times (13.5 + 80)}\,\frac{175^2 + 96^2}{(175 \times 96)} = 2670 \text{ kips-in}$$

This moment M_D is only an estimate of the actual moment that will exist between the beam and column. The actual moment will depend on the strength of the beam-to-column connection. The strength of the beam-to-column connection can be assessed by considering the forces induced in the connection by the moment M_D as shown in Fig. 2.12. The distortional force F_D is assumed to act as shown through the gusset edge connection centroids. If the brace force P is a tension, the angle between the beam and column tends to decrease, compressing the gusset between them, so F_D is a compression. If the brace force P is a compression, the angle between the beam and column tends to increase and F_D is a tension. Figure 2.12 shows how the distortional force F_D is

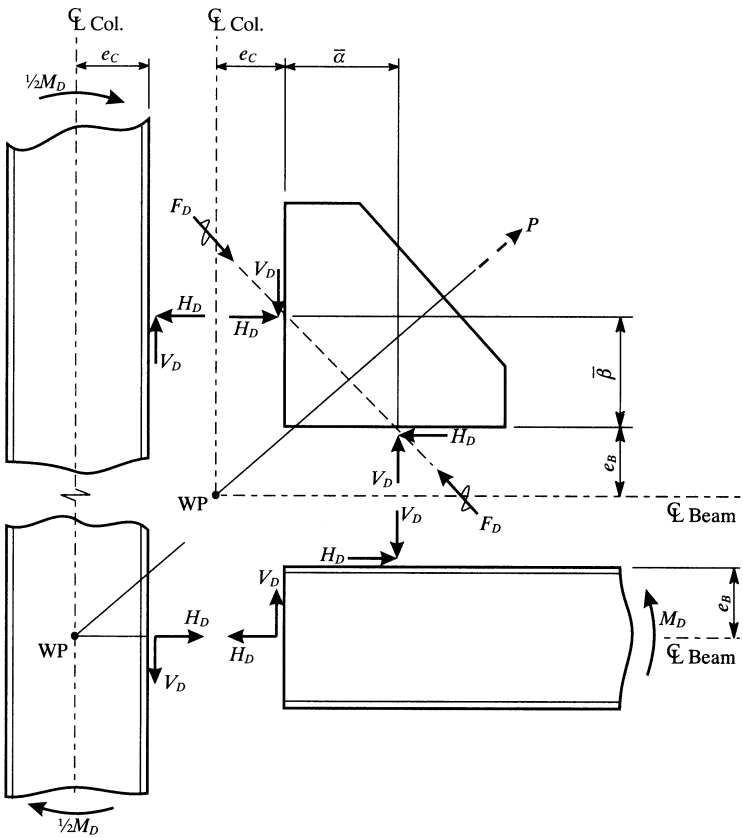


Figure 2.12 Distribution of distortion forces.

distributed throughout the connection. From Fig. 2.12, the following relationships exist between F_D, its components H_D and V_D, and M_D:

$$F_D = \sqrt{H_D^2 + V_D^2}$$

$$\bar{\beta} H_D = \bar{\alpha} V_D$$

$$H_D = \frac{M_D}{(\bar{\beta} + e_B)}$$

For the elastic case with no angular distortion

$$H_D = \frac{2670}{(12.25 + 12.05)} = 110 \text{ kips}$$

$$V_D = \frac{\bar{\beta}}{\bar{\alpha}} H_D = \frac{12.25}{15} \times 110 = 89.8 \text{ kips}$$

It should be remembered that these are just estimates of the distortional forces. The actual distortional forces will be dependent also upon the strength of the connection. But it can be seen that these estimated distortional forces are not insignificant. Compare, for instance, H_D to H_C. H_C is 218 kips tension when H_D is 110 kips compression. The net axial design force would then be 218 − 110 = 108 kips rather than 218 kips.

The strength of the connection can be determined by considering the strength of each interface, including the effects of the distortional forces. The following interface forces can be determined from Figs. 2.3 and 2.12.

For the gusset-to-beam interface:

$$T_B(\text{tangential force}) = H_B + H_D$$

$$N_B(\text{normal force}) = V_B - V_D$$

For the gusset-to-column interface:

$$T_C = V_C + V_D$$

$$N_C = H_C - H_D$$

For the beam-to-column interface:

$$T_{BC} = |V_B - V_D| + R$$

$$N_{BC} = |H_C - H_D| \pm A$$

The only departure from a simple equilibrium solution to the bracing connection design problem was in the assumption that frame action would allow the beam-to-column connection to be designed for an axial force equal to the maximum of H_c and A, or max (218, 150) = 218 kips. Thus, the design shown in Fig. 2.8 has its beam-to-column connection designed for N_{BC} = 218 kips and T_{BC} = 170 kips. Hence

$$N_{BC} = |218 - H_D| + 150 = 218$$

means that H_D = 150 kips and

$$V_D = \frac{12.25}{15} \times 150 = 122.5 \text{ kips}$$

From

$$T_{BC} = |V_\mathrm{B} - 122.5| + 170 = 170$$

$$V_B = 122.5 \text{ kips}$$

Note that in order to maintain the beam to column loads of 170 kips shear and 218 kips tension, the gusset-to-beam-shear V_B must increase from 0 to 122.5 kips. Figure 2.13 shows the transition from the original load distribution to the final distribution as given in Fig. 2.13*d*. Note also that N_{BC}

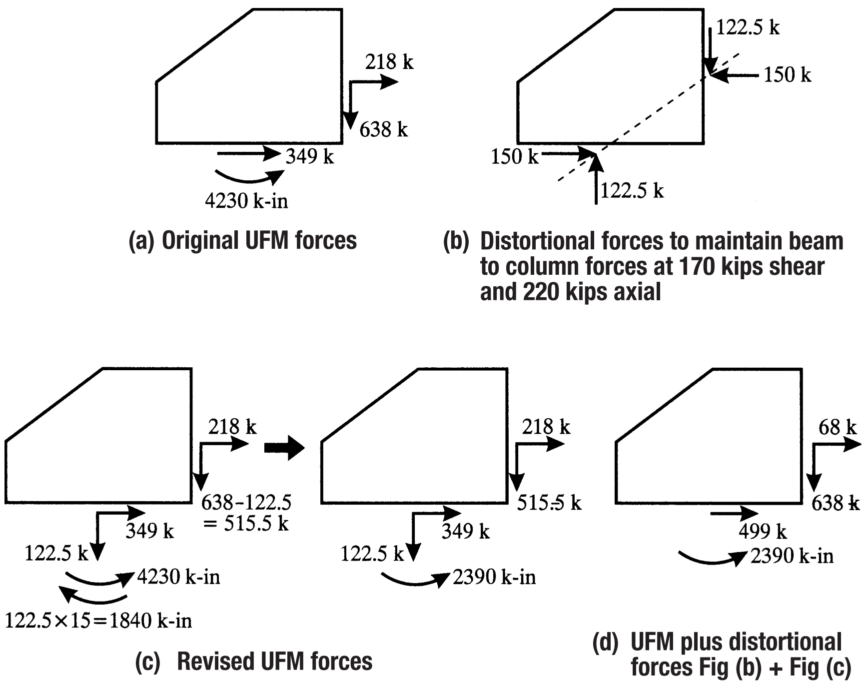


Figure 2.13 Admissible combining of UFM and distortional forces.

could have been set as $17.1 \times 14 = 239$ kips, rather than 218 kips, because this is the axial capacity of the connection at 170 kips shear. The N_{BC} value of 218 kips is used to cover the case when there is no excess capacity in the beam-to-column connection. Now, the gusset-to-beam and gusset-to-column interfaces will be checked for the redistributed loads of Fig. 2.13*d*.

Gusset to beam.

1. *Gusset stresses*:

$$f_v = \frac{499}{1.5 \times 30} = 11.1 \text{ ksi} < 21.6 \text{ ksi, ok}$$

$$f_b = \frac{2390 \times 4}{1.5 \times 30^2} = 7.08 \text{ ksi} < 32.4 \text{ ksi, ok}$$

2. *Weld of gusset to beam flange*:

$$f_R = \sqrt{11.1^2 + 7.08^2} \times \frac{1.5}{2} = 9.87$$

$$\Theta = \tan^{-1}(7.08/11.1)$$

$$= 32.5°$$

$$\mu = 1 + 0.5 \sin^{1.5} (32.5)$$
$$= 1.20°$$
$$D = \frac{9.85}{1.392 \times 1.20} = 5.90$$

A $^3/_8$-in fillet weld is indicated, which is less than what was provided. No ductility factor is used here because the loads include a redistribution.

Gusset to column. This connection is ok without calculations because the loads of Fig. 2.13d are no greater than the original loads of Fig. 2.13*a*.

Discussion. From the foregoing analysis, it can be seen that the AISC-suggested procedure for the beam-to-column connection, where the actual normal force

$$N_{BC} = |H_C - H_D| \pm \mathrm{A}$$

is replaced by

$$N_{BC} = \max\,(H_c, A)$$

is justified.

It has been shown that the connection is strong enough to carry the distortional forces of Fig. 2.13*b*, which are larger than the elastic distortional forces.

In general, the entire connection could be designed for the combined UFM forces and distortional forces, as shown in Fig. 2.13*d* for this example. This set of forces is also admissible. The UFM forces are admissible because they are in equilibrium with the applied forces. The distortional forces are in equilibrium with zero external forces. Under each set of forces, the parts of the connection are also in equilibrium. Therefore, the sum of the two loadings is admissible because each individual loading is admissible. A safe design is thus guaranteed by the lower bound theorem of limit analysis. The difficulty is in determining the distortional forces. The elastic distortional forces could be used, but they are only an estimate of the true distortional forces. The distortional forces depend as much on the properties of the connection, which are inherently inelastic and affect the maintenance of the angle between the members, as on the properties and lengths of the members of the frame. For this example, the distortional forces are $[(150 - 110)/110] \times 100 = 36\%$ greater than the elastic distortional forces. In full-scale tests by Gross (1990) as reported by Thornton (1991), the distortional forces were about $2^1/_2$ times the elastic distortional forces while the overall frame remained elastic. Because of the difficulty in establishing values for the distortional forces, and because the UFM has been shown to be conservative when they are ignored (Thornton, 1991, 1995b), they are not included in bracing connection design, except implicitly as noted here to justify replacing $|H_C - H_D| \pm A$ with max (H_C, A).

2.2.1.6 Load paths have consquences. The UFM produces a load path that is consistent with the gusset plate boundaries. For instance, if the gusset-to-column connection is to a column web, no horizontal force is directed perpendicular to the column web because unless it is stiffened, the web will not be able to sustain this force. This is clearly shown in the physical test results of Gross (1990) where it was reported that bracing connections to column webs were unable to mobilize the column weak axis stiffness because of web flexibility.

A mistake that is often made in connection design is to assume a load path for a part of the connection, and then to fail to follow through to make the assumed load path capable of carrying the loads (satisfying the limit states). Note that load paths include not just connection elements, but also the members to which they are attached. As an example, consider the connection of Fig. 2.14*a*. This is a configuration similar to that of Fig. 2.1*b* with minimal transfer force into and out of the braced bay.

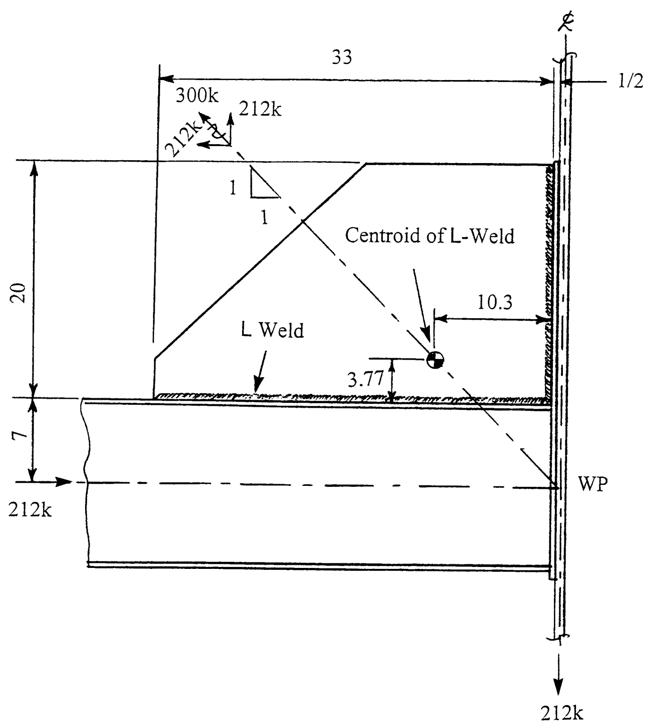


Figure 2.14*a* Bracing connection to demonstrate the consequences of an assumed load path.

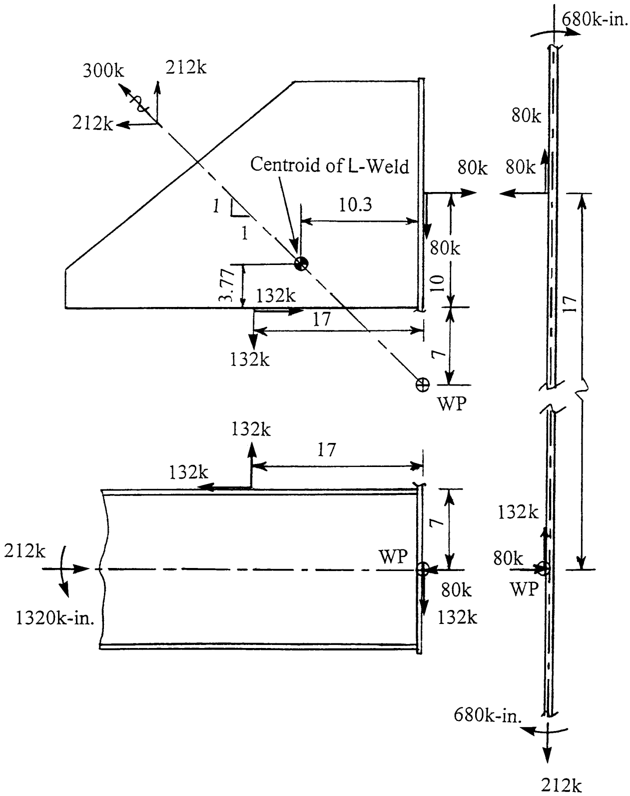


Figure 2.14*b* Free body diagrams for L weld method.

It is proposed to consider the welds of the gusset to the beam flange and to the $^1/_2$-in end plate as a single L-shaped weld. This will be called the L weld method, and is similar to model 4, the parallel force method, which is discussed by Thornton (1991). This is an apparently perfectly acceptable proposal and will result in very small welds because the centroid of the weld group will lie on or near the line of action of the brace. In the example of Fig. 2.14*a*, the geometry is arranged to cause the weld centroid to lie exactly on the line of action to simplify the calculation. This makes the weld uniformly loaded, and the force per inch is $f = 300/(33 + 20) = 5.66$ kips/in in a direction parallel to the brace line of action, which has horizontal and vertical components of $5.66 \times 0.7071 = 4.00$ kips/in. This results in free-body diagrams for the gusset, beam, and column as shown in Fig. 2.14*b*. Imagine how difficult it would be to obtain the forces on the free-body diagram of the gusset and other members if the weld were not uniformly loaded! Every inch of the weld would have a force of different magnitude and direction. Note that while the gusset is in equilibrium under the parallel forces alone, the beam

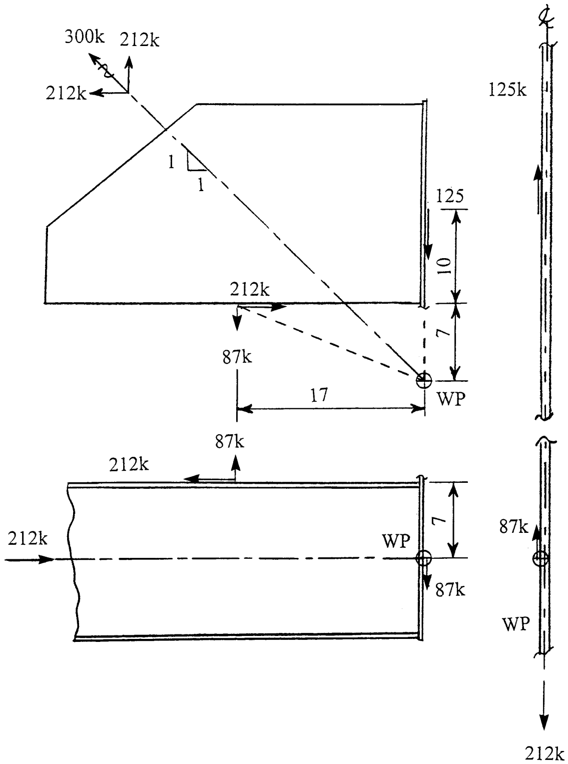


Figure 2.14*c* Free body diagrams for uniform force method.

and the column require the moments as shown to provide equilibrium. For comparison, the free-body diagrams for the UFM are given in Fig. 2.14*c*. These forces are always easy to obtain and no moments are required in the beam or column to satisfy equilibrium.

From the unit force $f = 5.66$ kips/in, the gusset-to-beam and gusset-to-end plate weld sizes are $D = 5.66/(2 \times 1.392) = 2.03$ sixteenths, actual required size. For comparison, the gusset-to-beam weld for the UFM would be

$$D = \frac{\sqrt{87^2 + 212^2}}{2 \times 33 \times 1.392} \times 1.25 = 3.12$$

actual required size, a 54% increase over the L weld method weld of $D = 2.03$. While the L weld method weld is very small, as expected with this method, now consider the load paths through the rest of the connection.

Gusset to column.

Bolts. The bolts are A325N-$^7/_8$-in. diameter. with $\phi r_v = 21.6$ kips and $\phi r_t = 40.6$ kips. The shear per bolt is $80/12 = 6.67$ kips < 21.6 kips, ok. The

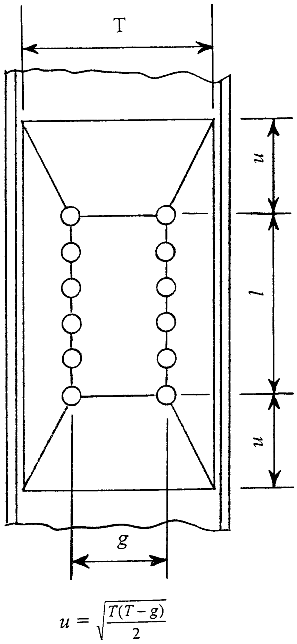


Figure 2.14*d* Deformation method for yield-line analysis of column web.

tension per bolt is 80/12 = 6.67 kips, but ϕr_t must be reduced due to interaction. Thus

$$\phi r_t{}' = 0.75\left[1.3 \times 90 - \left(\frac{90}{0.75 \times 48}\right)\left(\frac{6.67}{0.601}\right)\right]0.601$$
$$= 40.2 \text{ kips} < 40.6 \text{ kips}$$

so use $\phi r_t' = 40.6$ kips Since 40.6 > 6.67, the bolts are ok for shear and tension.

End plate. This involves the standard prying action calculations as follows:

$$b = (5.5 - 0.375)/2 = 2.56, a = (8 - 5.5)/2 = 1.25 < 1.25\text{b}$$

so use

$$a = 1.25; b' = 2.56 - 0.875/2 = 2.12, a' = 1.25 + 0.875/2$$
$$= 1.69, \rho = b'/a' = 1.25, \delta = 1 - 0.9375/3 = 0.69, p = 3;$$
$$t_c = \sqrt{((4.44 \times 40.6 \times 2.12/(3 \times 58))} = 1.48;$$

try an end plate 1/2 in thick.

Calculate

$$\alpha' = \frac{1}{0.698(1 + 1.25)}\left[\left(\frac{1.48}{0.5}\right)^2 - 1\right] = 4.94$$

Since $\alpha' > 1$, use $\alpha' = 1$, and the design tension strength is

$$T_d = 40.6 \times \left(\frac{0.5}{1.48}\right)^2 \times 1.69 = 7.83 \text{ kips} \geq 6.67 \text{ kips, ok}$$

The $\frac{1}{2}$-in end plate is ok.

Column web. The column web sees a transverse force of 80 kips. Figure 2.14d shows a yield-line analysis (Anand and Bertz, 1981) of the column web. The normal force ultimate strength of the yield pattern shown is

$$P_u = 8m_p \left\{ \sqrt{\frac{2T}{T - g}} + \frac{l}{2(T - g)} \right\}$$

where $m_p = 1/4\, F_y t_w^2$. For the present problem, $m_p = 0.25 \times 50 \times (0.44)^2 = 2.42$ kips-in/in, $T = 11.25$ in g $= 5.5$ in and $l = 15$ in so

$$P_u = 8 \times 2.42 \left(\sqrt{\frac{2 \times 11.25}{(11.25 - 5.5)}} + \frac{15}{2(11.25 - 5.5)} \right) = 63.5 \text{ kips}$$

Thus $\phi P_u = 0.9 \times 63.5 = 57.2$ kips < 80 kips, no good, and the column web is unable to sustain the horizontal force from the gusset without stiffening or a column web-doubler plate. Figure 2.15 shows a possible stiffening arrangement.

It should be noted that the yield-line pattern of Fig. 2.14*d* compromises the foregoing end plate/prying action calculation. That analysis assumed double curvature with a prying force at the toes of the end plate a distance a from the bolt lines. But the column web will bend away as shown in Fig. 2.14*d* and the prying force will not develop. Thus, single curvature bending in the end plate must be assumed, and the required end plate thickness is given by AISC, 2005, p. 9–10.

$$t_{\text{req}} = \sqrt{\frac{4.44Tb'}{pF_u}} = \sqrt{\frac{4.44 \times 6.67 \times 2.12}{3 \times 58}} = 0.600 \text{ in}$$

and a $\frac{5}{8}$-in-thick end plate is required.

Gusset to beam. The weld is already designed. The beam must be checked for web yield and crippling, and web shear.

Web yield. $\phi R_{wy} = 10 \times 0.305 \times 50\,(32 + 2.5 \times 1.12) = 531$ kips $>$ 132 kips, ok

Web crippling.

$$\phi R_{\text{wcp}} = 0.75 \times 0.8 \times 0.305^2 \left[1 + \left(\frac{32}{13.7}\right)\left(\frac{0.305}{0.530}\right)^{1.5} \right] \sqrt{\frac{50 \times 29000 \times 0.530}{0.305}}$$
$$= 179 \text{ kips} > 132 \text{ kips, ok}$$

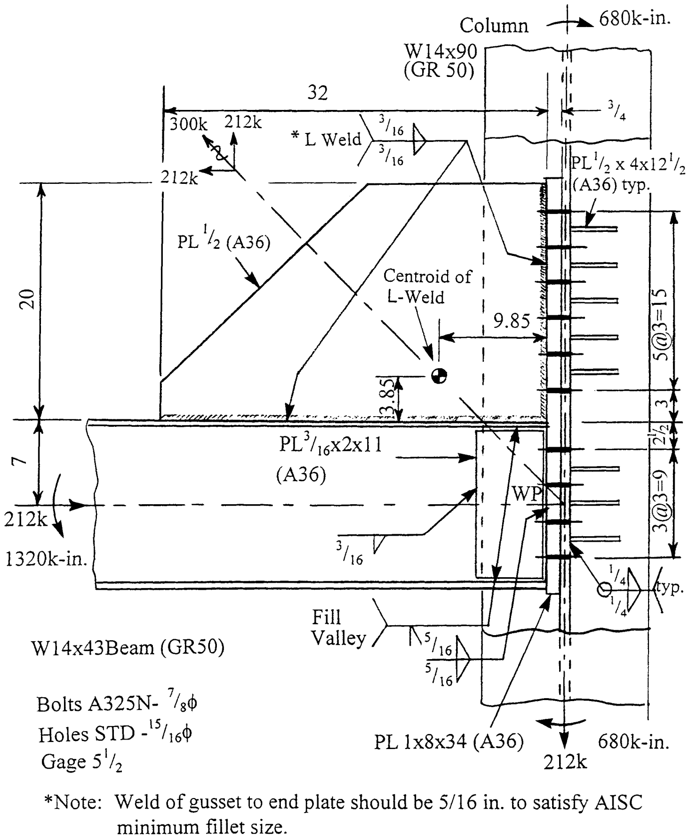


Figure 2.15 Design by L weld method.

Web shear. The 132-kip vertical load between the gusset and the beam flange is transmitted to the beam-to-column connection by the beam web. The shear design strength is

$$\phi R_{vw} = 1.0 \times 0.6 \times 0.305 \times 13.7 \times 50 = 125 \text{ kips} < 132 \text{ kips, no good}$$

To carry this much shear, a web-doubler plate is required. Starting at the toe of the gusset plate, 132/33 = 4.00 kips of shear is added per inch. The doubler must start at a distance x from the toe where $4.00x = 125$, $x = 31.0$ in. Therefore, a doubler of length $34 - 31.0 = 2$ in is required, measured from the face of the end plate. The doubler thickness t_d required is $1.0 \times 0.6 \times 50 \times (t_d + 0.305) \times 13.4 = 132$, $t_d = 0.02$ in, so use a minimum thickness $^3/_{16}$-in plate of grade 50 steel. If some yielding before ultimate load is reached is acceptable, grade 36 plate can be used. The thickness required would be $t_d = 0.02 \times 50/36 = 0.028$ in so a $^3/_{16}$-in A36 plate is also ok.

Beam to column. The fourth connection interface (the first interface is the brace-to-gusset connection, not considered here), the beam-to-column, is the most heavily loaded of them all. The 80 kips horizontal between the gusset and column must be brought back into the beam through this connection to make up the beam (strut) load of 212 kips axial. This connection also sees the 132 kips vertical load from the gusset-to-beam connection.

Bolts. The shear per bolt is 132/8 = 16.5 kips < 21.6 kips, ok. The reduced tension design strength is

$$\phi r_t' = 0.75 \left[1.3 \times 90 - \left(\frac{90}{0.75 \times 48} \right) \left(\frac{16.5}{0.601} \right) \right] 0.601$$

$$= 21.8 \text{ kips} < 40.6 \text{ kips},$$

so use $\phi r_t' = 21.8$ kips. Since 21.8 kips > 80/8 = 10.0 kips, the bolts are ok for tension and shear.

End Plate. As discussed for the gusset-to-column connection, there will be no prying action and hence double curvature in the end plate, so the required end plate thickness is

$$t_{\text{req}} = \sqrt{\frac{4.44 \times 10.0 \times 2.12}{3 \times 58}} = 0.736 \text{ in}$$

A $^3/_4$-in end plate is required. This plate will be run up to form the gusset-to-column connection, so the entire end plate is a 3/4-in plate (A36).

Column web. Using the yield-line analysis for the gusset-to-column connection, $T = 11.25$, $g = 5.5$, $l = 9$

$$\phi P_u = 0.9 \times 8 \times 2.42 \left[\sqrt{\frac{2 \times 11.25}{5.75}} + \frac{9}{2 \times 5.75} \right]$$

$$= 48 \text{ kips} < 80 \text{ kips, no good}$$

Again, the column web must be stiffened as shown in Fig. 2.15, or a doubler must be used.

Stiffener. If stiffeners are used, the most highly loaded one will carry the equivalent tension load of three bolts or 30.0 kips to the column flanges. The stiffener is treated as a simply supported beam $12\frac{1}{2}$ in long loaded at the gage lines. Figure 2.15 shows the arrangement. The shear in the stiffener is 30.0/2 = 15.0 kips, and the moment is 15.0 × (12.5 − 5.5)/2 = 52.5 kips-in. Try a stiffener of A36 steel 1/2 × 4:

$$f_v = \frac{15.0}{0.5 \times 4} = 7.50 \text{ ksi} < 21.6 \text{ ksi, ok}$$

$$f_b = \frac{52.5 \times 4}{0.5 \times 4^2} = 26.3 \text{ ksi} < 32.4 \text{ ksi, ok}$$

The 1/2 × 4 stiffener is ok. Check buckling, $b/t = 4/0.5 = 8 < 15$, ok.

Weld of stiffener to column web. Assume about 3 in of weld at each gage line is effective, that is $1.5 \times 1 \times 2 = 3$. Then

$$D = \frac{10.0 + 0.5 \times 10.0}{2 \times 3 \times 1.5 \times 1.392} = 1.19 \qquad \text{use } \frac{3}{16} \text{ fillet welds}$$

Weld of stiffener to column flange.

$$D = \frac{15.0}{2 \times (4 - 0.75) \times 1.392} = 1.66 \qquad \text{use } \frac{3}{16} \text{ fillet welds}$$

Weld of end plate to beam web and doubler plate. The doubler is 3/16 in thick and the web is 0.305 in thick, so $0.1875/0.4925 = 0.38$ or 38% of the load goes to the doubler and 42% goes to the web. The load is $\sqrt{132^2 + 80^2} = 154$ kips. The length of the weld is $13.66 - 2 \times 0.530 = 12.6$ in. The weld size to the doubler is $D = 0.38 \times 154/(2 \times 12.6 \times 1.392) = 1.67$ and that to the web is $D = 0.42 \times 154/(2 \times 12.6 \times 1.392) = 1.84$, so 3/16 in minimum fillets are indicated.

Additional discussion. The 80-kip horizontal force between the gusset and the column must be transferred to the beam-to-column connections. Therefore, the column section must be capable of making this transfer. The weak axis shear capacity (design strength) of the column is

$$\phi R_v = 1.0 \times 0.6 \times 50 \times 0.710 \times 14.5 \times 2 = 618 \text{ kips} > 80 \text{ kips, ok}$$

It was noted earlier that the column and the beam require couples to be in equilibrium. These couples could act on the gusset-to-column and gusset-to-beam interfaces, since they are free vectors, but this would totally change these connections. Figure 2.14*b* shows them acting in the members instead, because this is consistent with the L weld method. For the column, the moment is $80 \times 17 = 1360$ kips-in and is shown with half above and half below the connection. The bending strength of the column is $\phi M_{py} = 0.9 \times 50 \times 133 = 5985$ kips-in so the $1360/2 = 680$ kips-in is 11% of the capacity, which probably does not seriously reduce the column's weak axis bending strength. For the beam, the moment is $132 \times 17 - 132 \times 7 = 1320$ kips-in (should be equal and opposite to the column moment since the connection is concentric—the slight difference is due to numerical roundoff). The bending strength of the beam is $\phi M_{px} = 0.9 \times 50 \times 69.6 = 3146$ kips-in so the 1320 kips-in couple uses up 42% of the beam's bending strength. This will greatly reduce its capacity to carry 212 kips in compression and is probably not acceptable.

This completes the design of the connection by the L weld method. The reader can clearly see how the loads filter through the connection, that is, the load paths involved. The final connection as shown in Fig. 2.15 has small welds of the gusset to the beam and the end plate, but the rest of the connection is very expensive. The column stiffeners are expensive, and also compromise any connections to the opposite side of the column web. The $^3/_4$-in end plate must be flame cut because it is generally too thick for most shops to shear. The web-doubler plate is an expensive detail and involves welding in the beam k-line area, which may be prone to cracking (AISC, 1997). Finally, although the connection is satisfactory, its internal admissible force distribution that satisfies equilibrium requires generally unacceptable couples in the members framed by the connection.

As a comparison, consider the design that is achieved by the UFM. The statically admissible force distribution for this connection is given in Fig. 2.14*c*. Note that all elements (gusset, beam, and column) are in equilibrium with no couples. Note also how easily these internal forces are computed. The final design for this method, which can be verified by the reader, is shown in Fig. 2.16. There is no question that this connection is less expensive than its L weld counterpart in Fig. 2.15, and it does not compromise the strength of the column and strut. To summarize, the L weld method seems a good idea at the outset, but a complete "trip" through the load paths ultimately exposes it as a fraud, that is, it produces expensive and unacceptable connections. As a final comment, a load path assumed for part of a connection affects every other part of the connection, including the members that frame to the connection.

2.2.1.7 Bracing connections utilizing shear plates. All of the bracing connection examples presented here have involved connections to the column using end plates or double clips, or are direct welded. The UFM is not limited to these attachment methods. Figures 2.17 and 2.18 show connections to a column flange and web, respectively, using shear plates. These connections are much easier to erect than the double-angle or shear plate type because the beams can be brought into place laterally and easily pinned. For the column web connection of Fig. 2.18, there are no common bolts that enhance erection safety. The connections shown were used on an actual job and were designed for the tensile strength of the brace to resist seismic loads in a ductile manner.

2.2.1.8 Connections with non-concentric work points. The UFM can be easily generalized to this case as shown in Fig. 2.19*a*, where x and y locate the specified non-concentric work point (WP) from the intersection of the beam and column flanges. All of the forces on the connection interfaces are the same as for the concentric UFM, except that there is an extra moment on the gusset plate $M = Pe$, which can be applied to

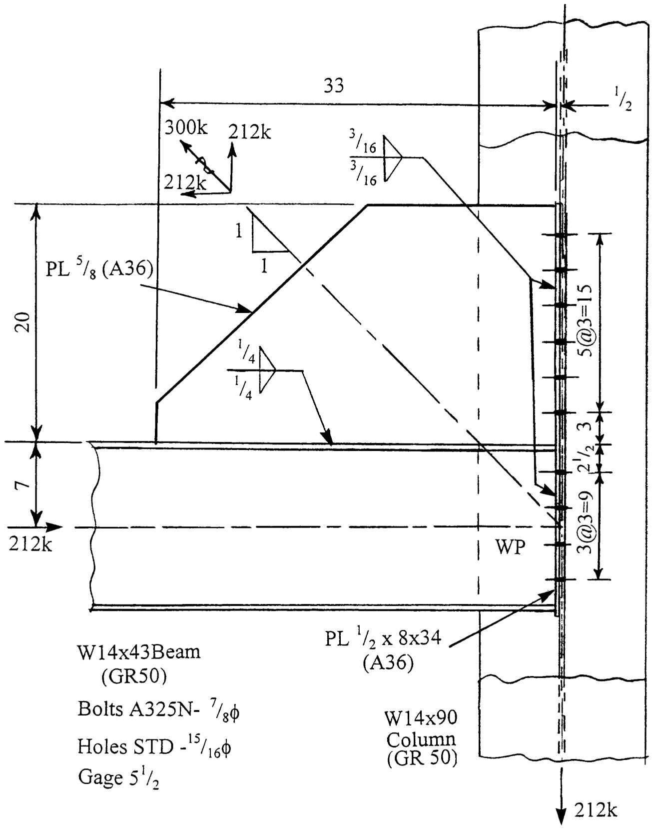


Figure 2.16 Design by uniform force method.

the stiffer gusset edge. It should be noted that this non-concentric force distribution is consistent with the findings of Richard (1986), who found very little effect on the force distribution in the connection when the work point is moved from concentric to non-concentric locations. It should also be noted that a non-concentric work point location induces a moment in the structure of $M = Pe$, and this may need to be considered in the design of the frame members. In the case of Fig. 2.19*a*, since the moment $M = Pe$ is assumed to act on the gusset-to-beam interface, it must also be assumed to act on the beam outside of the connection, as shown. In the case of a connection to a column web, this will be the actual distribution (Gross, 1990), unless the connection to the column mobilizes the flanges, as for instance is done in Fig. 2.15 by means of stiffeners.

An alternate analysis, where the joint is considered rigid, that is, a connection to a column flange, the moment M is distributed to the beam

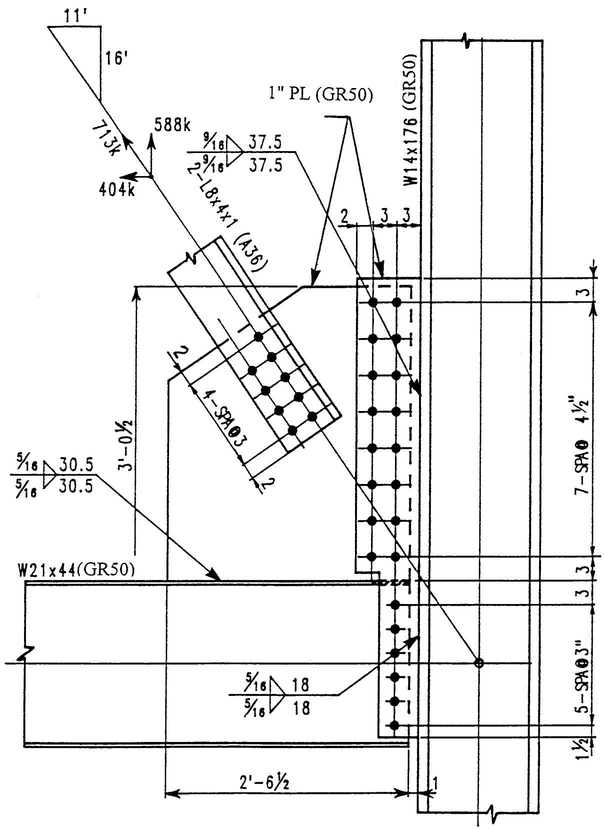


Figure 2.17 Bracing connection to a column flange utilizing a shear plate.

and column in accordance with their stiffnesses (the brace is usually assumed to remain an axial force member and so is not included in the moment distribution), can be performed. If η denotes the fraction of the moment that is distributed to the beam, then horizontal and vertical forces, H' and V', respectively, acting at the gusset to beam, gusset-to-column, and beam-to-column connection centroids due to the distribution of M are

$$H' = \frac{(1 - \eta)M}{\bar{\beta} + e_B}$$

$$V' = \frac{M - H'\bar{\beta}}{\bar{\alpha}}$$

These forces, shown in Fig. 2.19b, are to be added algebraically to the concentric UFM forces acting at the three connection interfaces. Note

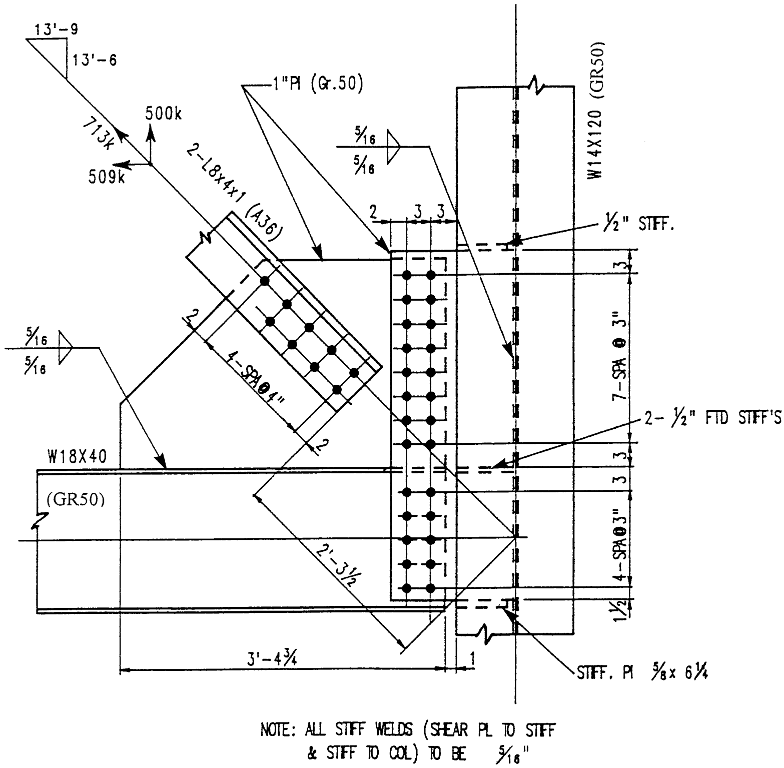


Figure 2.18 Bracing connection to a column web utilizing a shear plate.

that for connections to column webs, $\eta = 1, H' = 0$, and $V' = M/\bar{\alpha}$, unless the gusset-to-column web and beam-to-column web connections positively engage the column flanges, as for instance in Fig. 2.15.

Example. Consider the connection of Sec. 2.2.1.4 as shown in Fig. 2.8, but consider that the brace line of action passes through the corner of the gusset rather than to the gravity axis intersection of the beam and the column. Using the data of Fig. 2.8, $e_C = 8.37$, $e_B = 12.05$, $\bar{\alpha} = 15.0$, $\beta = 12.25$,

$$\theta = \tan^{-1}\left[\left(10\frac{11}{16}\right)/12\right] = 41.7^\circ$$

Since the specified work point is at the gusset corner, $x = y = 0$, and $e = 12.05 \sin 41.7^\circ - 8.37 \cos 41.7^\circ = 1.76$ in. Thus, $M = Pe = 855 \times 1.77 = 1510$ kips-in and using the frame data of Sec. 2.2.1.5,

$$\eta = 13.5/(13.5 + 80) = 0.144$$

$$H' = \frac{(1 - 0.144)1510}{(12.25 + 12.05)} = 53 \text{ kips}$$

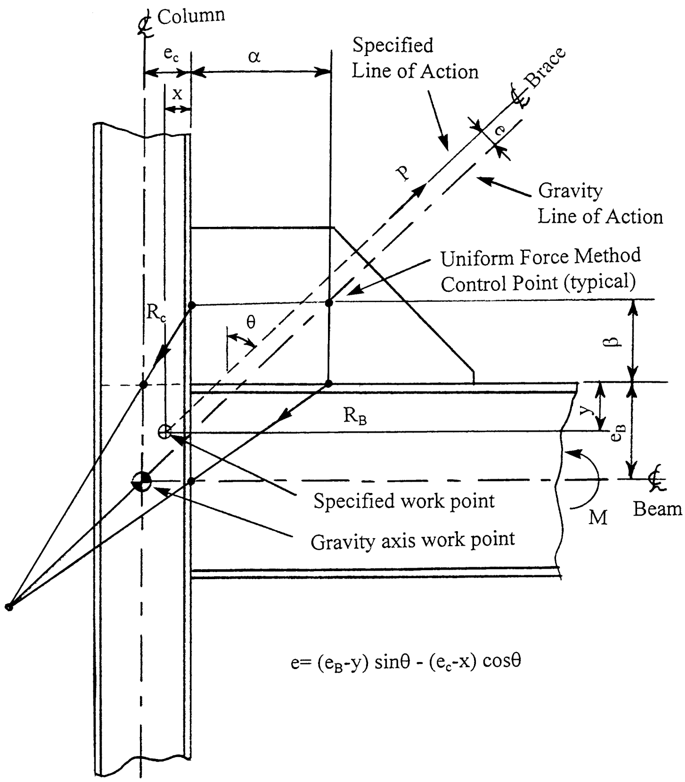


Figure 2.19*a* Nonconcentric uniform force method.

$$V' = \frac{1510 - 53 \times 12.25}{15} = 58\,\text{kips}$$

These forces are shown on the gusset in Fig. 2.19*c*. This figure also shows the original UFM forces of Fig. 2.13*a*. The design of this connection will proceed in the same manner as shown in Sec. 2.2.1.4, but the algebraic sum of the original forces and the additional forces due to the non-concentric work point are used on each interface.

2.2.2 Truss connections

2.2.2.1 Introduction. The UFM as originally formulated can be applied to trusses as well as to bracing connections. After all, a vertical bracing system is just a truss as seen in Fig. 2.1, which shows various arrangements. But bracing systems generally involve orthogonal members, whereas trusses, especially roof trusses, often have a sloping top chord. In order to handle this situation, the UFM has been generalized as shown

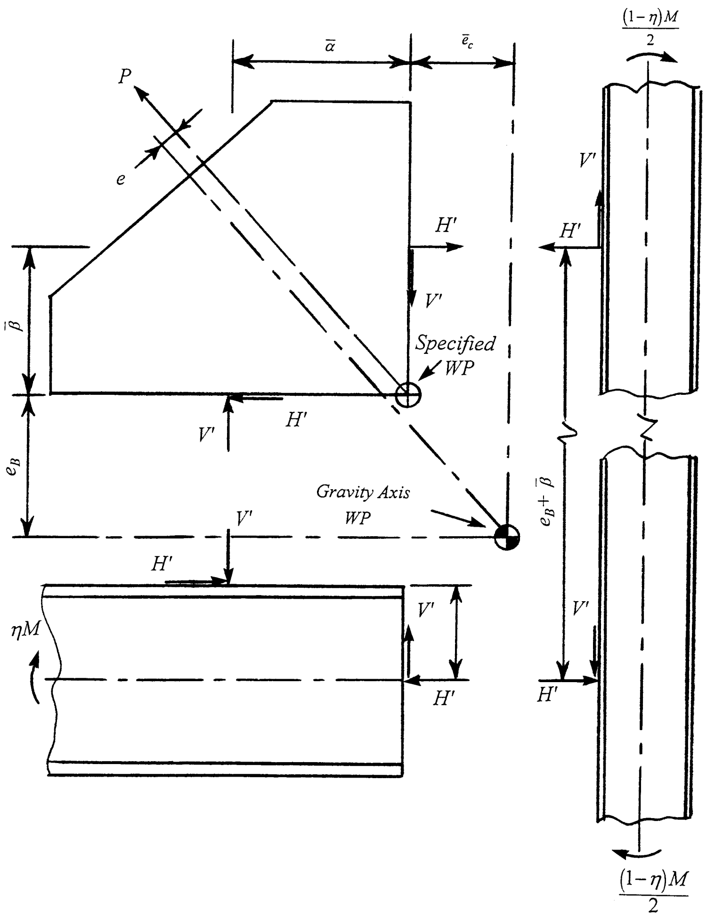


Figure 2.19*b* Extra forces due to nonconcentric work point.

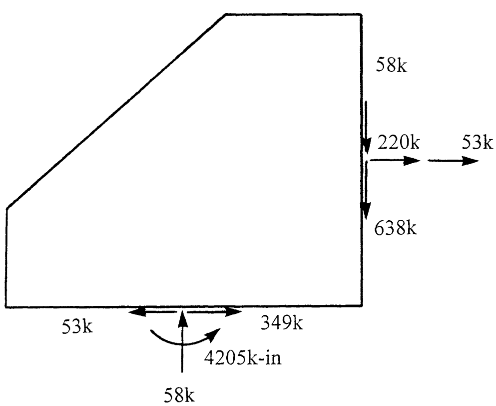
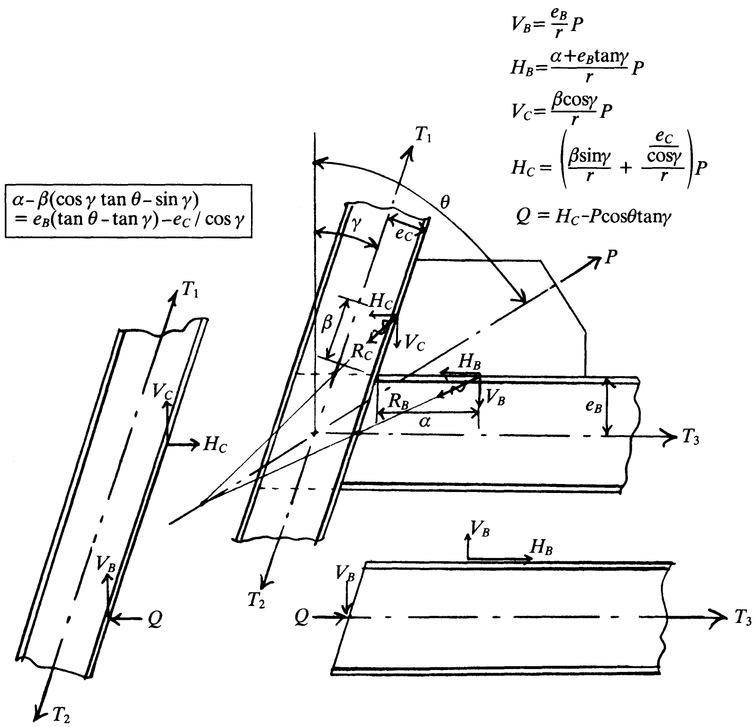


Figure 2.19*c* Uniform force method and nonconcentric forces combined.



$$r = \sqrt{\left(\alpha + e_B\tan\gamma + \beta\sin\gamma + \frac{e_C}{\cos\gamma}\right)^2 + (e_B + \beta\cos\gamma)^2}$$

Figure 2.20 Generalized uniform force method.

in Fig. 2.20 to include nonorthogonal members. As before, α and β locate the centroids of the gusset edge connections and must satisfy the constraint shown in the box on Fig. 2.20. This can always be arranged when designing a connection, but in checking a given connection designed by some other method, the constraint may not be satisfied. The result is gusset edge couples, which must be considered in the design.

2.2.2.2 A numerical example. As an application of the UFM to a truss, consider the situation of Fig. 2.21. This is a top chord connection in a large aircraft hangar structure. The truss is cantilevered from a core support area. Thus, the top chord is in tension. The design shown in Fig. 2.21 was obtained by generalizing the KISS method (Thornton, 1995b) shown in Fig. 2.22 for orthogonal members to the nonorthogonal case. The KISS method is the simplest admissible design method for truss and bracing connections. On the negative side, however, it generates large, expensive,

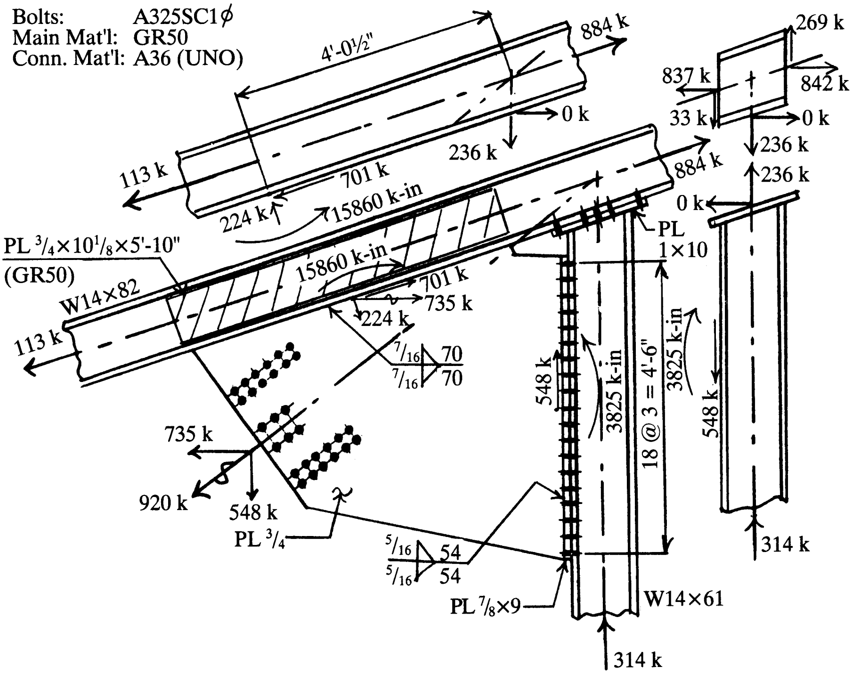


Figure 2.21 KISS method—gusset forces arc brace components.

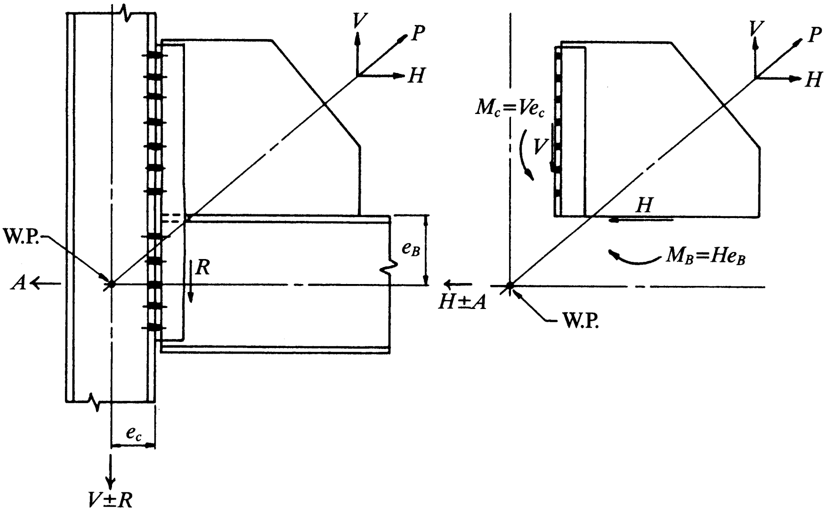


Figure 2.22 The KISS method.

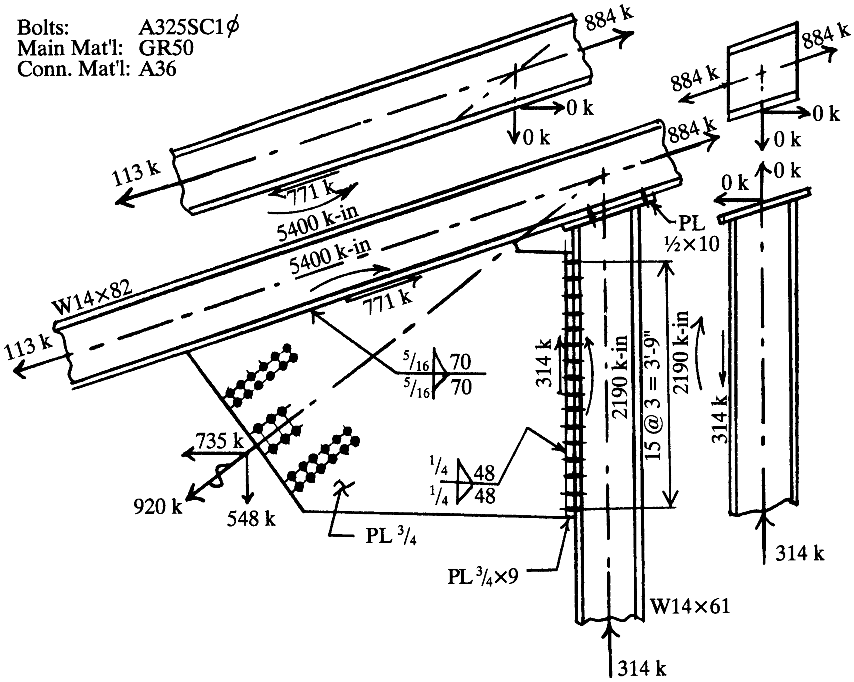


Figure 2.23 KISS method—brace components are tangent to gusset edges.

and unsightly connections. The problem with the KISS method is the couples required on the gusset edges to satisfy equilibrium of all parts. In the Fig. 2.21 version of the KISS method, the truss diagonal, horizontal, and vertical components are placed at the gusset edge centroids as shown. The couples 15,860 kips-in on the top edge and 3825 kips-in on the vertical edge are necessary for equilibrium of the gusset, top chord, and truss vertical, with the latter two experiencing only axial forces away from the connection. It is these couples that require the 3/4-in chord doubler plate, the 7/16-in fillets between the gusset and chord, and the 38-bolt 7/8-in end plate on the vertical edge.

The design shown in Fig. 2.23 is also obtained by the KISS method with the brace force resolved into tangential components on the gusset edges. Couples still result, but are much smaller than in Fig. 2.21. The resulting connection requires no chord doubler plate, 5/16-in fillets of the gusset to the chord, and a 32-bolt 3/4-in end plate on the vertical edge. This design is much improved over that of Fig. 2.21.

When the UFM of Fig. 2.20 is applied to this problem, the resulting design is as shown in Fig. 2.24. The vertical connection has been reduced to only 14 bolts and a 1/2-in end plate.

The designs of Figs. 2.21, 2.23, and 2.24 are all satisfactory for some admissible force system. For instance, the design of Fig. 2.21 will be

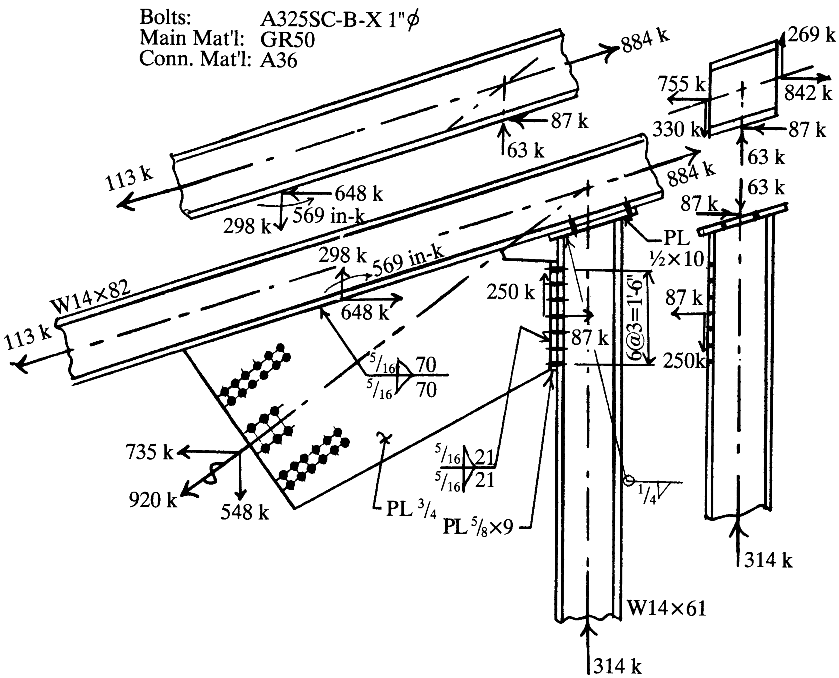


Figure 2.24 Uniform force method.

satisfactory for the force systems of Figs. 2.23 and 2.24, and the design of Fig. 2.23 will be satisfactory for the force system of Fig. 2.24. How can it be determined which is the "right" or "best" admissible force system to use? The lower bound theorem of limit analysis provides an answer. This theorem basically says that for a given connection configuration, that is, Fig. 2.21, 2.23, or 2.24, the statically admissible force distribution that maximizes the capacity of the connection is closest to the true force distribution. As a converse to this, for a given load, the smallest connection satisfying the limit states is closest to the true required connection. Of the three admissible force distributions given in Figs. 2.21, 2.23, and 2.24, the distribution of Fig. 2.24, based on the UFM, is the "best" or "right" distribution.

2.2.2.3 A numerical example. To demonstrate the calculations required to design the connections of Figs. 2.21, 2.23, and 2.24, for the statically admissible forces of these figures, consider for instance the UFM forces and the resulting connection of Fig. 2.24.

The geometry of Fig. 2.24 is arrived at by trial and error. First, the brace-to-gusset connection is designed, and this establishes the minimum size of gusset. For calculations for this part of the connection, see Sec. 2.2.1.2. Normally, the gusset is squared off as shown in Fig. 2.23, which gives 16 rows of bolts in the gusset-to-truss vertical connection. The gusset-to-top chord connection is pretty well constrained by geometry to be about 70 in long plus about $13\frac{1}{2}$ in for the cutout. Starting from the configuration of Fig. 2.23, the UFM forces are calculated from the formulas of Fig. 2.20 and the design is checked. It will be found that Fig. 2.23 is a satisfactory design via the UFM, even though it fails via the KISS method forces of Fig. 2.21. Although the gusset-to-top chord connection cannot be reduced in length because of geometry, the gusset-to-truss vertical is subject to no such constraint. Therefore, the number of rows of bolts in the gusset-to-truss vertical is sequentially reduced until failure occurs. The last-achieved successful design is the final design as shown in Fig. 2.24.

The calculations for Fig. 2.24 and the intermediate designs and the initial design of Fig. 2.23 are performed in the following manner. The given data for all cases are:

$$P = 920 \text{ kips}$$
$$e_B = 7 \text{ in}$$
$$e_c = 7 \text{ in}$$
$$\gamma = 17.7^\circ$$
$$\theta = 36.7^\circ$$

The relationship between α and β is

$$\alpha - \beta(0.9527 \times 0.7454 - 0.3040) = 7(0.7454 - 0.3191) - 7/0.9527$$
$$\alpha - 0.4061\beta = -4.363$$

This relationship must be satisfied for these to be no couples on the gusset edges. For the configuration of Fig. 2.24 with seven rows of bolts in the gusset-to-truss vertical connection (which is considered the gusset-to-beam connection of Fig. 2.20) $\overline{\alpha} = 18.0$ in. Then,

$$\beta = \frac{18 + 4.363}{0.4061} = 55.07 \text{ in}$$

From Fig. 2.24, the centroid of the gusset to top chord (which is the gusset-to-column connection of Fig. 2.20) is $\overline{\beta} = 13.5 + 70/2 = 48.5$ in. Since $\overline{\beta} \neq \beta$, there will be a couple on this edge unless the gusset geometry is adjusted to make $\overline{\beta} = \beta = 55.07$. In this case, we will leave the gusset geometry unchanged and work with the couple on gusset-to-top chord interface.

Rather than choosing $\overline{\alpha} = 18.0$ in, we could have chosen $\overline{\beta} = 48.5$ and solved for $\alpha \neq \overline{\alpha}$. In this case, a couple will be required on the gusset-to truss vertical interface unless gusset geometry is changed to make $\overline{\alpha} = \alpha$.

Of the two possible choices, the first is the better one because the rigidity of the gusset-to-top chord interface is much greater than that of the gusset-to-truss vertical interface. This is so because the gusset is direct welded to the center of the top chord flange and is backed up by the chord web, whereas the gusset-to-truss vertical involves a flexible end plate and the bending flexibility of the flange of the truss vertical. Thus, any couple required to put the gusset in equilibrium will tend to migrate to the stiffer gusset-to-top chord interface.

With $\alpha = 18.0$ and $\beta = 55.07$,

$$\begin{aligned} r &= [(18.0 + 7 \times 0.3191 + 55.07 \times 0.3040 + 7/0.9527)^2 \\ &\quad + (7 + 55.07 \times 0.9527)^2]^{1/2} \\ &= 74.16 \text{ in} \end{aligned}$$

and from the equations of Fig. 2.20,

$$\begin{aligned} V_C &= 648 \text{ kips} \\ H_C &= 298 \text{ kips} \\ V_B &= 87 \text{ kips} \\ H_B &= 250 \text{ kips} \end{aligned}$$

For subsequent calculations, it is necessary to convert the gusset-to-top chord forces to normal and tangential forces as follows: the tangential or shearing component is

$$T_C = V_C \cos\gamma + H_C \sin\gamma = (\beta + e_C \tan\gamma)\frac{P}{r}$$

The normal or axial component is

$$N_C = H_C \cos\gamma - V_C \sin\gamma = \frac{e_C P}{r}$$

The couple on the gusset-to-top chord interface is then

$$M_C = |N_C(\beta - \bar{\beta})|$$

Thus

$$\begin{aligned} T_C &= (55.07 + 7 \times .3191)\frac{920}{74.16} = 711^{\mathrm{k}} \\ N_C &= 7 \times \frac{920}{74.16} = 86.6^{\mathrm{k}} \\ M_C &= 86.6 \times (55.07 - 48.5) = 569 \text{ kips-in} \end{aligned}$$

Each of the connection interfaces will now be designed.

1. *Gusset to top chord.*

   a. *Weld*: Weld length is 70 in.

$$f_v = \frac{711}{2 \times 70} = 5.08\,\text{kips/in}$$

$$f_a = \frac{86.6}{2 \times 70} = 0.61\,\text{kips/in}$$

$$f_b = \frac{569 \times 2}{70^2} = 0.23\,\text{kips/in}$$

$$f_R = \sqrt{(5.08)^2 + (0.61 + 0.23)^2} = 5.1\,\text{kips/in}$$

$$D = \frac{5.1}{1.392} = 3.7\ 16^{\text{ths}}$$

   Check ductility

$$f_{\text{ave}} = \sqrt{5.08^2 + \left(0.61 + \frac{0.23}{2}\right)^2} = 5.1\,\text{kips/in}$$

   Since $1.25 \times f_{\text{ave}} = 6.4 > 5.1$, size weld for ductility requirement

$$D = \frac{6.4}{1.392} = 4.60$$

   Use $^5/_{16}$ fillet weld.

   b. *Gusset stress:*

$$f_v = \frac{711}{0.75 \times 70} = 13.5\,\text{ksi} < 21.6\,\text{ksi, ok}$$

$$f_a + f_b = \frac{86.6}{0.75 \times 70} + \frac{569 \times 4}{0.75 \times 70^2} = 2.27\,\text{ksi} < 32.4\,\text{ksi, ok}$$

   c. *Top chord web yield:* The normal force between the gusset and the top chord is $T_c = 86.6$ kips and the couple is $M_c = 569$ kips-in. The contact length N is 70 in. The couple M_c is statically equivalent to equal and opposite normal forces $V_s = M_c/(N/2) = 569/35 = 16.2$ kips. The normal force V_s acts over a contact length of $N/2 = 35$ in. For convenience, an equivalent normal force acting over the contact length N can be defined as

$$N_{\text{Cequiv.}} = N_C + 2 \times V_s = 86.6 + 2 \times 16.2 = 119\text{ kips}$$

   Now, for web yielding

$$\phi R_{wy} = 1.0 \times (5k + N)\,F_{yw} t_w = 1.0 \times (5 \times 1.625 + 70) \times 50 \times 0.510$$
$$= 1992\text{ kips} > 119\text{ kips, ok}$$

d. Top chord web crippling:

$$\phi R_{\text{wcp}} = 0.75 \times 0.8 \times t_w^2\left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}}$$

$$= 0.75 \times 0.8 \times .510^2\left[1 + 3\left(\frac{70}{14.3}\left(\frac{0.510}{0.855}\right)^{1.5}\right)\right]$$

$$\times \sqrt{\frac{29000(50)(0.855)}{0.510}} = 1890 \text{ kips} > 119 \text{ kips, ok}$$

In the web crippling check, the formula used is that for a location greater than d/2 from the chord end because $\bar{\beta} = 13.5 + 70/2 = 48.5$ in $> 14.31/2 = 7.2$ in. $\bar{\beta}$ is the position of the equivalent normal force. Additionally, the restraint provided by the beam end connection is sufficient to justify the check away from the end of the beam.

The checks for web yield and crippling could have been dismissed by inspection in this case, but were completed to illustrate the method. Another check that should be made when there is a couple acting on a gusset edge is to ensure that the transverse shear induced on the supporting member, in this case the top chord W14 × 82, can be sustained. In this case, the induced transverse shear is $V_s = 16.2$ kips. The shear capacity of the W14 × 82 is $0.510 \times 14.3 \times 1.0 \times 0.6 \times 50 = 219$ kips > 16.2 kips, ok. Now consider for contrast, the couple of 15,860 kips-in shown in Fig. 2.21. For this couple, $V_s = 15860/35 = 453$ kips > 219 kips, so a 3/4 in. doubler plate of GR50 steel is required as shown in Fig. 2.21.

2. *Gusset to truss vertical:*

a. Weld:

$$f_v = \frac{250}{2.21} = 5.95 \text{ kips/in}$$

$$f_a = \frac{87}{2 \times 21} = 2.07 \text{ kips/in}$$

$$f_R = \sqrt{5.95^2 + 2.07^2} = 6.30 \text{ kips/in}$$

$$\text{Fillet weld size required} = \frac{6.30}{1.392} = 4.5\ 16^{\text{ths}}$$

Because of the flexibility of the end plate and truss vertical flange, there is no need to size the weld to provide ductility. Therefore, use a 5/16 fillet weld.

b. Bolts and end plate: The bolts are A325SC-B-X, 1″ϕ in standard holes designed to the serviceability level, as is the default in the specification. The end plate is 9 in wide and the gage of the bolts is $5\frac{1}{2}$ in. Thus, using the prying action formulation notation of the

AISC 13th Manual (2005). The slip resistance per bolt can be calculated as

$$\phi R_n = \phi \mu D_u h_{sc} T_b = 1.0(0.50)(1.13)(1.0)(51) = 28.8 \text{ kips}$$

The tensile strength per bolt, excluding prying, from Table 7-2 is 53.0 kips.

$$b = \frac{5.5 - 0.75}{2} = 2.375$$

$$a = \frac{9 - 5.5}{2} = 1.75 < 1.25 \times 2.375 \text{ ok}$$

$$b' = 2.375 - 0.5 = 1.875$$

$$a' = 1.75 + 0.5 = 2.25$$

$$\rho = \frac{1.875}{2.25} = 0.833$$

$$\delta = 1 - \frac{1.0625}{3} = 0.646$$

Shear per bolt $= V = 250/14 = 17.9 < 28.8$ kips, ok. Tension per bolt $= T = 87/14 = 6.21$ kips. Thus

$$\phi r'_n = 1.13 \times 51 \times \left(1 - \frac{17.9}{28.8}\right) = 21.8 \text{ kips} > 6.21 \text{ kips, ok}$$

Try $\frac{1}{2}$ plate

$$t_c = \sqrt{\frac{4.44 \times 21.8 \times 1.875}{3 \times 58}} = 1.02$$

$$\alpha' = \frac{1}{0.646 \times 1.833}\left[\left(\frac{1.02}{0.5}\right)^2 - 1\right] = 2.67$$

Use $\alpha' = 1$

$$T_d = 21.8\left(\frac{0.5}{1.02}\right)^2 \times 1.833 = 9.60 \text{ kips} > 6.21 \text{ kips ok}$$

Use the $\frac{1}{2}$-in plate for the end plate.

c. *Truss vertical flange:* The flange thickness of the W14 × 61 is 0.645 in which exceeds the end plate thickness as well as being Grade 50 steel. The truss vertical flange is therefore, ok by inspection, but a calculation will be performed to demonstrate how the flange can be checked. A formula (Mann and Morris, 1979) for an effective bolt pitch can be derived from yield-line analysis as

$$p_{\text{eff}} = \frac{p(n - 1) + \pi\bar{b} + 2\bar{a}}{n}$$

where the terms are as previously defined in Fig. 2.11. For the present case

$$\bar{b} = \frac{5.5 - 0.375}{2} = 2.5625$$

$$\bar{a} = \frac{10 - 5.5}{2} = 2.25$$

$$n = 7$$

$$p = 3$$

$$p_{\text{eff}} = \frac{3 \times (7 - 1) + \pi \times 2.5625 + 2 \times 2.25}{7} = 4.36$$

Once p_{eff} is determined, the prying action theory of the AISC Manual is applied.

$$b = \bar{b} = 2.5625$$

$$b' = 2.5625 - 0.5 = 2.0625$$

a = smaller of $\bar{a}$ and a for the end plate $= 1.75 < 1.25 \times 2.5625$ ok

$$a' = 1.75 + 0.5 = 2.25$$

$$\rho = \frac{b'}{a'} = 0.917$$

$$\delta = 1 - 1.0625/4.36 = 0.756$$

$$t_c = \sqrt{\frac{4.44 \times 21.8 \times 2.0625}{4.36 \times 65}} = 0.839 \text{ in}$$

$$\alpha' = \frac{1}{0.756 \times 1.917}\left[\left(\frac{0.839}{0.645}\right)^2 - 1\right] = 0.477$$

$$T_d = 21.8\left(\frac{0.645}{0.839}\right)^2 \times (1 + 0.477 \times 0.756)$$

$$= 17.5 \text{ kips} > 6.21 \text{ kips, ok}$$

3. *Truss vertical-to-top chord connection:*

The forces on this connection, from Figs. 2.20 and 2.24 are

$$\text{Vertical} = Q = 298 - 920 \times \cos(36.7) \times \tan(17.7) = 63 \text{ kips}$$

$$\text{Horizontal} = 87 \text{ kips}$$

Converting these into normal and tangential components

$$T_{BC} = 87 \cos\gamma - 63 \sin\gamma = 64 \text{ kips}$$

$$N_{BC} = 87 \sin\gamma + 63 \cos\gamma = 86 \text{ kips (compression)}$$

a. *Bolts:* Since the normal force is always compression, the bolts see only the tangential or shear force; thus, the number of bolts required is

$$\frac{64}{28.8} = 2.2 \text{ use 4 bolts}$$

b. *Weld:* Use a profile fillet weld of the cap plate to the truss vertical, but only the weld to the web of the vertical is effective because there are no stiffeners between the flanges of the top chord. Thus, the effective length of weld is

$$(13.89 - 2 \times 0.645)/\cos\gamma = 13.23 \text{ in}$$

$$f_v = \frac{64}{2 \times 13.23} = 2.42 \text{ kips/in}$$

$$f_a = \frac{86}{2 \times 13.23} = 3.25 \text{ kips/in}$$

$$f_R = \sqrt{2.42^2 + 3.25^2} = 4.05 \text{ kips/in}$$

The weld size required is $\dfrac{4.05}{1.392} = 2.91$

Use ¼ (AISC minmum size)

Check the W14 × 61 web to support required 2.91 16ths FW. For welds of size W on both sides of a web of thickness t_w

$$1.0 \times 0.6 \times F_y t_w \geq 0.75 \times 0.60 \times 70 \times 0.7071 \times \mathrm{W} \times 2$$

or

$$t_w \geq 1.48 \text{ W for grade 50 steel}$$

Thus for W = 2.91/16 = 0.182

$$t_{w_{\min}} = 1.48 \times 0.182 = 0.269 \text{ in}$$

Since the web thickness of a W14 × 61 is 0.375, the web can support the welds.

c. *Cap plate:*

The cap plate thickness will be governed by bearing. The bearing design strength per bolt is

$$\phi r_p = 0.75 \times 2.4 \times 58 \times t_p \times 1$$

The load per bolt is 64/4 = 16.0 kips. The required cap plate thickness is thus

$$t_p = \frac{16.0}{0.75 \times 2.4 \times 58 \times 1} = 0.153$$

Use a $\frac{1}{2}$-in cap plate.

This completes the calculations required to produce the connection of Fig. 2.24.

2.2.3 Hanger connections

The most interesting of the genre is the type that involves prying action, sometimes of both the connection fitting and the supporting member. Figure 2.25 shows a typical example. The calculations to determine the capacity of this connection are as follows: The connection can be broken into three main parts, that is, the angles, the piece W16 × 57, and the supporting member, the W18 × 50. The three main parts are joined by two additional parts, the bolts of the angles to the piece W16 and the bolts from the piece W16 to the W18. The load path in this connection is unique. The load P passes from the angles through the bolts into the piece W16, thence through bolts again into the supporting W18. The latter bolt group is arranged to straddle the brace line of action. These bolts then see only direct tension and shear, and no additional tension due to moment. Statics is sufficient to establish this. Consider now the determination of the capacity of this connection.

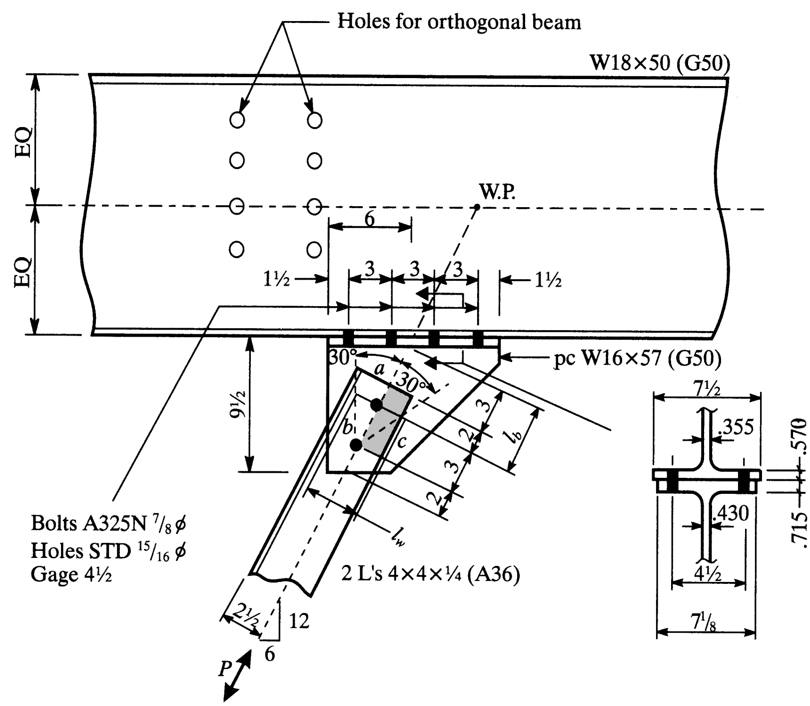


Figure 2.25 Typical bolted hanger connection.

1. *Angles:* The limit states for the angles are gross tension, net tension, block shear rupture, and bearing. The load can be compression as well as tension in this example. Compression will affect the angle design, but tension will control the above limit states.

   a. Gross tension: The gross area A_{gt} is $1.94 \times 2 = 3.88$ in^2. The capacity (design strength) is

   $$\phi R_{gt} = 0.9 \times 36 \times 3.88 = 126 \text{ kips}$$

   b. Net tension: The net tension area is $A_{nt} = 3.88 - 0.25 \times 1.0 \times 2 = 3.38$ in^2. The effective net tension area A_e is less than the net area because of shear lag since only one of the two angle legs is connected. From the AISC Specification (2005) Section D3.3

   $$U = \max(0.60, 1 - 1.09/3) = 0.637$$
   $$A_e = U \times A_{nt} = 0.637 \times 3.38 = 2.15$$

   The net tension capacity is

   $$\phi R_{nt} = 0.75 \times 58 \times 2.15 = 93.5 \text{ kips}$$

   c. Block shear rupture: This failure mode involves the tearing out of the cross-hatched block in Fig. 2.25. The failure is by yield on the longitudinal line through the bolts (line ab) and a simultaneous fracture failure on the perpendicular line from the bolts longitudinal line to the angle toe (line bc).

   Because yield on the longitudinal section may sometimes exceed fracture on this section, the AISC Specification J4.3 limits the strength to the lesser of the two. Thus, the block shear limit state is

   $$\phi R_{bs} = 0.75[U_{bs} F_u A_{nt} + \min\{0.6 F_y A_{gv}, 0.6 F_u A_{nv}\}]$$

   where the terms will be defined in the following paragraphs.

   For line ab, the gross shear area is

   $$A_{gv} = 5 \times 0.25 \times 2 = 2.5 \text{ in}^2$$

   and the net shear area is

   $$A_{nv} = 2.5 - (1.5 \times 0.25 \times 1.0)2 = 1.75 \text{ in}^2$$

   For line bc, the gross tension area is

   $$A_{gt} = 1.5 \times 0.25 \times 2 = 0.75 \text{ in}^2$$

   and the net tension area is

   $$A_{nt} = 0.75 - 0.5 \times 1.0 \times 0.25 \times 2 = 0.5 \text{ in}^2$$

The term U_{bs} accounts for the fact that for highly eccentric connections, the tension force distribution on section bc will not be uniform. In this case, U_{bs} is taken as 0.5. In the present case, the force distribution is essentially uniform because the angle gage line and the angle gravity axis are close to each other. Thus $U_{bs} = 1.0$, and the block shear strength is

$$\phi R_{bs} = 0.75[1.0 \times 58 \times 0.5 + \min\{0.6 \times 36 \times 2.5, 0.6 \times 58 \times 1.75\}]$$
$$= 62.2 \text{ kips.}$$

d. Shear / bearing / tearout on bolts and parts:
Bearing, tearout, and bolt shear are inextricably tied to each bolt. Therefore, it is no longer possible to check bolt shear for the bolt group as a whole, and bearing/tearout for each part separately, and then to take the minimum of these limit states as the controlling limit state. The procedure is as follows for each bolt. For the upper bolt, the limit states are

1. bolt shear $\phi R_v = 0.75 \times 48 \times \pi/4 \times 0.875^2 \times 2 = 43.3$ kips
2. bearing on angles $\phi R_p = 0.75 \times 2.4 \times 0.875 \times 2 \times 0.25 \times 5 = 45.7$ kips
3. bearing on W16 × 57 $\phi R_p = 0.75 \times 2.4 \times 0.875 \times 0.43 \times 65 = 44.0$ kips
4. tearout on angles $\phi R_{to} = 0.75 \times 1.2(2 - 0.5 \times 0.9375) \times 2 \times 0.25 \times 58 = 40.0$ kips
5. tearout on W16 × 57 $\phi R_{to} = 0.7\ 5 \times 1.2(3 - 0.9375) \times 0.430 \times 65 = 51.9$ kips

The shear/bearing/tearout of the upper bolt is thus 40.0 kips.
For the lower bolt, the limit states are

1. bolt shear $\phi R_v = 43.4$ kips
2. bearing on the angles $\phi R_p = 45.7$ kips
3. bearing on the W16 × 57 $\phi R_p = 44.0$ kips
4. tearout on the angles $\phi R_{to} = 0.75 \times 1.2(3 - 0.9375) \times 2 \times 0.25 \times 58 = 53.8$ kips
5. tearout on the W15 × 57 $\phi R_{to} = 0.75 \times 1.2(2 - 0.5 \times 0.9375) \times 0.430 \times 65 = 38.5$ kips

The shear/bearing/tearout strength of the lower bolt is thus 38.5 kips, and the capacity of the connection in these limit states is $(R_{vp} = 40.0 \times 1 + 38.5 \times 1 = 78.5$ kips.

2. *Bolts—angles to piece W16:* The limit state for the bolts is shear. The shear capacity of one bolt is $\phi r_v = 0.75 \times 48 \times \pi/4 \times 0.875^2 = 21.6$ kips.

In this case, the bolts are in double shear and the double shear value per bolt is 21.6 × 2 = 43.3 kips/bolt. Note that because of bearing limitations, this value cannot be achieved. The bolt shear strength is limited by the bearing strength of the parts; thus the bolt shear strength is equal to the bearing strength, so

$$\phi R_y = \phi R_p = 78.5 \text{ kips}$$

3. *Piece W16 × 57:* The limit states for this part of the connection are Whitmore section yield and buckling, bearing, and prying action in conjunction with the W16 flange to W18 flange bolts. Because there is only one line of bolts, block shear is not a limit state. Bearing has already been considered with the angle checks.

   a. Whitmore section: This is the section denoted by l_w on Fig. 2.25. It is formed by 30° lines from the bolt furthest away from the end of the brace to the intersection of these lines with a line through and perpendicular to the bolt nearest to the end of the brace. Whitmore (1952) determined that this 30° spread gave an accurate estimate of the stress in gusset plates at the end of the brace. The length of the Whitmore section $l_w = 3(\tan 30°)2 = 3.46$ in.

   (1) Whitmore yield:

$$\phi R_{wy} = 0.9 \times 50 \times 3.46 \times 0.430 = 67.0 \text{ kips}$$

   where 0.430 is the web thickness of a W16 × 57.

   (2) Whitmore buckling:
   Tests (Gross, 1990; Dowswell, 2006) have shown that the Whitmore section can be used as a conservative estimate for gusset buckling. In the present case, the web of the W16 × 57 is a gusset. If the load P is a compression, it is possible for the gusset to buckle laterally in a sidesway mode. For this mode of buckling, the K factor is 1.2. The buckling length is $l_b = 5$ in in Fig. 2.25. Thus the slenderness ratio is

$$\frac{Kl}{r} = \frac{1.2 \times 5 \times \sqrt{12}}{0.430} = 48.3$$

   Since $Kl/r > 25$, Section J4.4 on strength of elements in compression does not apply; the column buckling equations of Chapter E apply. Thus, from Section E3,

$$F_e = \frac{\pi^2 \times 29000}{(28.3)^2} = 123$$

$$F_{cr} = \left[0.658^{\left(\frac{50}{123}\right)}\right]50 = 42.2 \text{ ksi}$$

$$\phi F_{cr} = 0.9 \times 42.2 = 38.0 \text{ ksi}$$

   and $\phi R_{wb} = 38.0 \times 3.46 \times 0.430 = 56.5 \text{ kips}$

b. Bearing: This has been considered with the angles, above.

c. Prying action: Prying action explicitly refers to the extra tensile force in bolts that connect flexible plates or flanges subjected to loads normal to the flanges. For this reason, prying action involves not only the bolts but the flange thickness, bolt pitch and gage, and in general, the geometry of the entire connection.

The AISC LRFD Manual presents a method to calculate the effects of prying. This method was originally developed by Struik (1969) and presented in the book (Kulak et al., 1987). The form used in the AISC LRFD Manual was developed by Thornton (1985), for ease of calculation and to provide optimum results, that is, maximum capacity for a given connection (analysis) and minimum required thickness for a given load (design). Thornton (1992, 1997) has shown that this method gives a very conservative estimate of ultimate load and shows that very close estimates of ultimate load can be obtained by using the flange ultimate strength, F_u, in place of yield strength, F_y, in the prying action formulas. More recently, Swanson (2002) has confirmed Thornton's (1992, 1997) results with modern materials. For this reason, the AISC Manual now uses F_u in place of F_y in the prying action formulas. Note that the resistance factor, ϕ, used with the F_u is 0.90, because the flange failure mode is yielding with strain hardening rather than fracture.

From the foregoing calculations, the capacity (design strength) of this connection is 56.5 kips. Let us take this as the design load (required strength) and proceed to the prying calculations. The vertical component of 56.5 is 50.5 kips and the horizontal component is 25.3 kips. Thus, the shear per bolt is $V = 25.3/8 = 3.16$ kips and the tension per bolt is $T = 50.5/8 = 6.31$ kips. Since $3.16 < 21.6$, the bolts are ok for shear. Note that the bolts also need to be checked for bearing as was done for the angles. In this case, bearing is seen to be "ok by inspection." The interaction equation for A325 *N* bolts is

$$F_{nt}' = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v \le F_{nt}$$

were F_{nt} = bolt nominal tensile strength = 90 ksi

F_{nv} = bolt nominal shear strength = 48 ksi

$\phi = 0.75$

f_v = the required shear strength per bolt.

With $V = 3.16$ kips/bolt, $f_v = 3.16/0.6013 = 5.26$ ksi, and

$$F'_{nt} = 1.3 \times 90 - \frac{90}{0.75 \times 48} \times 5.26 = 104 \text{ ksi, use } F'_{nt} = 90 \text{ ksi.}$$

Now, the design tensile strength per bolt is

$\phi r_t' = 0.75 \times 90 \times 0.6013 = 40.6$ kips is greater than the required strength (or load) per bolt $T = 6.31$ kips, the bolts are ok.

Now, to check prying of the W16 piece, following the notation of the AISC Manual,

$$b = \frac{4.5 - 0.430}{2} = 2.035$$

$$a = \frac{7.125 - 4.5}{2} = 1.3125$$

Check that $a < 1.25\text{b} = 1.25 \times 2.035 = 2.544$. Since $a = 1.3125 < 2.544$, use $a = 1.3125$. If $a > 1.25\text{b}$, $a = 1.25\text{b}$ would be used.

$$b' = 2.035 - 0.875/2 = 1.598$$

$$a' = 1.3125 + 0.875/2 = 1.75$$

$$\rho = \frac{b'}{a'} = 0.91$$

$$p = 3$$

$$\delta = 1 - \text{d}'/\text{p} = 1 - 0.9375/3 = 0.6875$$

$$\alpha' = \frac{1}{\delta(1 + \rho)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right]$$

$$t_c = \sqrt{\frac{4.44(\phi r_t')b'}{pF_u}} = \sqrt{\frac{4.44 \times 40.6 \times 1.598}{3 \times 65}} = 1.215$$

$$\alpha' = \frac{1}{0.6875 \times 1.91}\left[\left(\frac{1.215}{0.715}\right)^2 - 1\right] = 1.44$$

Since $\alpha' > 1$, use $\alpha' = 1$ in subsequent calculations. $\alpha' = 1.44$ means that the bending of the W16 $\times$ 57 flange will be the controlling limit state. The bolts will not be critical, that is, the bolts will not limit the prying strength. The design tensile strength T_d per bolt including the flange strength is

$$T_d = \phi r_t'\left(\frac{t}{t_c}\right)^2(1 + \delta) = 40.6\left(\frac{0.715}{1.215}\right)^2 1.6875$$

$$= 23.7 \text{ kips} > 5.96 \text{ kips, ok}$$

The subscript d denotes “design” strength.

In addition to the prying check on the piece W16 $\times$ 57, a check should also be made on the flange of the W18 $\times$ 50 beam. A method for doing this was presented in Fig. 2.11. Thus,

$$\bar{b} = \frac{4.5 - 0.355}{2} = 2.073$$

$$\bar{a} = \frac{7.5 - 4.5}{2} = 1.50$$

$$n = 4$$
$$p = 3$$
$$p_{\text{eff}} = \frac{3(4-1) + \pi \times 2.073 + 2 \times 1.50}{4} = 4.63$$

Now, using the prying formulation from the AISC Manual,

$$b = \bar{b} = 2.073$$
$$a = 1.3125$$

Note that the prying lever arm is controlled by the narrower of the two flanges.

$$b' = 2.073 - 0.875/2 = 1.636$$
$$a' = 1.3125 + 0.875/2 = 1.75$$
$$\rho = 0.93$$
$$p = p_{\text{eff}} = 4.63$$
$$\delta = 1 - 0.9375/4.63 = 0.798$$
$$t_c = \sqrt{\frac{4.44 \times 40.6 \times 1.636}{4.63 \times 50}} = 0.990$$
$$\alpha' = \frac{1}{0.798 \times 1.93}\left[\left(\frac{0.990}{0.570}\right)^2 - 1\right] = 1.31$$

Use $\alpha' = 1$

$$T_d = 40.6\left(\frac{0.570}{0.990}\right)^2 1.798 = 24.2^{\text{k}} > 5.96^{\text{k}} \qquad \text{ok}$$

Additional checks on the W18 × 50 beam are for web yielding. Since $5k = 5 \times 1.25 = 6.25 > p = 3$, the web tributary to each bolt at the k distance exceeds the bolt spacing and thus $N = 9$.

$\phi R_{wy} = 1.0 \times (9 + 5 \times 1.25) \times 50 \times 0.355 = 271$ kips > 50.5 kips, ok, and for web crippling, web crippling occurs when the load is compression, thus $N = 12$, the length of the piece W16.

$$\phi R_{wcp} = 0.75 \times 0.80 \times 0.355^2\left[1 + 3\left(\frac{12}{18.0}\right)\left(\frac{0.355}{0.570}\right)^{1.5}\right]$$
$$\times \sqrt{\frac{29000 \times 50 \times 0.570}{0.355}} = 229 \text{ kips} > 50.5 \text{ kips, ok}$$

This completes the design calculations for this connection. A load path has been provided through every element of the connection. For this type of connection, the beam designer should make sure that the bottom flange is stabilized if P can be compressive. A transverse beam framing nearby as shown in Fig. 2.25 by the W18 × 50 web hole pattern, or a bottom flange stay (kicker), will provide stability.

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2.2.4 Column base plates

The geometry of a column base plate is shown in Fig. 2.26. The area of the base plate is $A_1 = B \times N$. The area of the pier that is concentric with A_1 is A_2. If the pier is not concentric with the base plate, only the portion that is concentric can be used for A_2. The design strength of the concrete in bearing is

$$\phi_c F_p = 0.6 \times 0.85 f'_c \sqrt{\frac{A_2}{A_1}}$$

where f'_c is the concrete compressive strength in ksi and

$$1 \le \sqrt{\frac{A_2}{A_1}} \le 2$$

The required bearing strength is

$$f_p = \frac{P}{A_1}$$

where P is the column load (factored) in kips. In terms of these variables, the required base plate thickness is

$$t_p = l\sqrt{\frac{2f_p}{\phi F_y}}$$

where $l = \max\{m, n, \lambda n'\}$

ϕF_y = base plate design strength = $0.9\,F_y$

$$m = \frac{N - 0.95d}{2}$$

$$n = \frac{B - 0.8b_f}{2}$$

$$n' = \frac{\sqrt{db_f}}{4}$$

$$\lambda = \frac{2\sqrt{x}}{1 + \sqrt{1 - x}} \le 1$$

$$x = \frac{4db_f}{(d + b_f)^2}\,\frac{f_p}{\phi_c F_p}$$

d = depth of column

b_f = flange width of column

For simplicity, λ can always be conservatively taken as unity. The formulation given here was developed by Thornton (1990a, 1990b) based on previous work by Murray (1983), Fling (1970), and Stockwell (1975). It is the method given in the AISC Manual (2005).

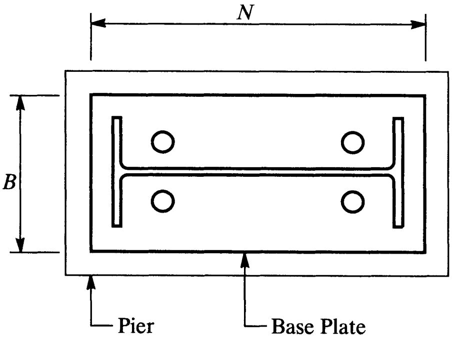


Figure 2.26 Column base plate.

Example. The column of Fig. 2.26 is a W24 × 84 carrying 600 kips. The concrete has $f'_c = 4.0$ ksi. Try a base plate of A36 steel, 4 in bigger than the column in both directions. Since $d = 24\,^1/_8$ and $b_f = 9$, $N = 24\,^1/_8 + 4 = 28\,^1/_8$, $b = 9 + 4 = 13$. Try a plate 28 × 13. Assume that 2 in of grout will be used, so the minimum pier size is 32 × 17. Thus $A_1 = 28 \times 13 = 364$ in², $A_2 = 32 \times 17 = 544$ in², $\sqrt{A_2/A_1} = 1.22 < 2$ (ok), and

$$\phi_c F_p = 0.6 \times 0.85 \times 4 \times 1.22 = 2.49 \text{ ksi}$$

$$f_p = \frac{600}{364} = 1.65 \text{ ksi} < 2.49 \text{ ksi, ok}$$

$$m = \frac{28 - 0.95 \times 24.125}{2} = 2.54$$

$$n = \frac{13 - 0.8 \times 9}{2} = 2.90$$

$$n' = \frac{\sqrt{24.125 \times 9}}{4} = 3.68$$

$$x = \frac{4 \times 24.125 \times 9.0}{(24.125 + 9.0)^2}\,\frac{1.65}{2.49} = 0.52$$

$$\lambda = \frac{2\sqrt{0.52}}{1 + \sqrt{1 - 0.52}} = 0.85$$

$$l = \max\{2.54, 2.90, 0.85 \times 3.68\} = 3.13$$

$$t_p = 3.13\sqrt{\frac{2 \times 1.65}{0.9 \times 36}} = 0.99 \text{ in}$$

Use a plate 1 × 13 × 28 of A36 steel. If the conservative assumption of $\lambda = 1$ were used, $t_p = 1.17$ in, which indicates a $1\,^1/_4$-in-thick base plate.

Erection considerations. In addition to designing a base plate for the column compression load, loads on base plates and anchor rods during erection should be considered. The latest OSHA requirements postulate a 300 lb. load 18 in off the column flange in the strong axis direction, and the same load 18 in off the flange tips in the weak axis direction. Note these loads would be applied sequentially. A common design load for erection, which is much more stringent than the OSHA load, is a 1-kip working load, applied at the top of the column in any horizontal direction. If the column is, say, 40 ft high, this 1-kip force at a lever arm of 40 ft will cause a significant couple at the base plate and anchor bolts. The base plate, anchor bolts, and column-to-base plate weld should be checked for this construction load condition. The paper by Murray (1983) gives some yield-line methods that can be used for doing this. Figure 2.26 shows four anchor rods. This is an OSHA erection requirement for all columns except minor posts.

2.2.5 Splices—columns and truss chords

Section J1.4 of the AISC Specification (2005) says that finished-to-bear compression splices in columns need be designed only to hold the parts "securely in place." For this reason, the AISC provides a series of "standard" column splices in the AISC *Manual of Steel Construction*. These splices are nominal in the sense that they are designed for no particular loads. Section J1.4 also requires that splices in trusses be designed for at least 50% of the design load (required compression strength), or for the moment and shear resulting from a transverse load equal to 2% of the required compressive strength of the member, whichever is less severe. The difference between columns and "other compression members," such as compression chords of trusses, is that for columns, splices are usually near lateral support points, such as floors, whereas trusses can have their splices at mid-panel points where there is no lateral support. Either the 50% requirement or the 2% requirement can be used to address this situation.

Column splices. Figure 2.27 shows a standard AISC column splice for a W14 $\times$ 99 to a W14 $\times$ 109. If the column load remains compression, the strong-axis column shear can be carried by friction. The coefficient of static friction of steel to steel is on the order of 0.5 to 0.7, so quite high shears can be carried by friction. Suppose the compression load on this column is 700 kips. How much major axis bending moment can this splice carry? Even though these splices are nominal, they can carry quite significant bending moment. The flange area of the W14 $\times$ 99 is $A_f = 0.780 \times 14.565 = 11.4$ in^2. Thus, the compression load per flange is $700 \times 11.4/29.1 = 274$ kips. In order for a bending moment to cause a tension in the column flange, this load of 274 kips must first be unloaded. Assuming that the flange force acts at the flange centroid, the moment in the column can be represented as:

$$M = T(d - t_f) = T(14.16 - 0.780) = 13.38T$$

If $T = 274$ kips, one flange will be unloaded, and $M = 13.38 \times 274 = 3666$ kips-in $= 306$ kips-ft. The design strength in bending for this column (assuming sufficient lateral support) is $\phi M_p = 647$ kips-ft. Thus, because of the compression load, the nominal AISC splice, while still seeing no load, can carry almost 50% of the column's bending capacity.

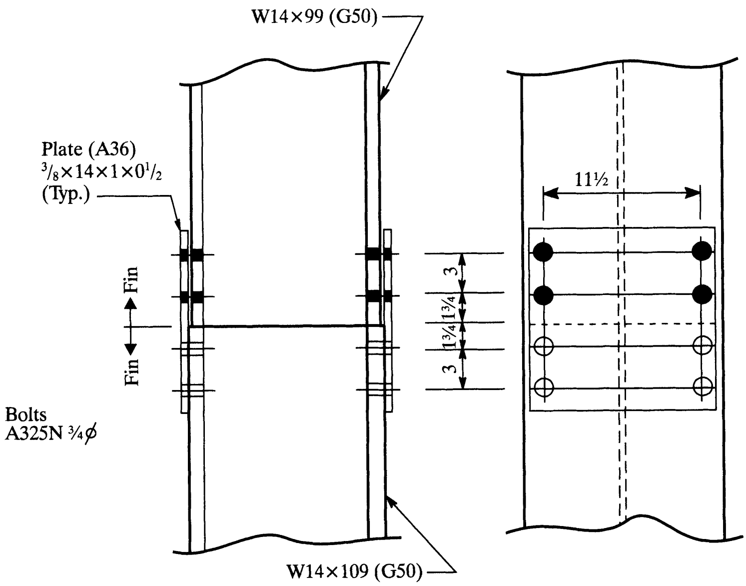


Figure 2.27 An AISC standard column splice.

The splice plates and bolts will allow additional moment to be carried. It can be shown that the controlling limit state for the splice material is bolt shear. For one bolt $\phi r_v = 15.9$ kips. Thus for 4 bolts $\phi R_v = 15.9 \times 4 = 63.6$ kips. The splice forces are assumed to act at the faying surface of the deeper member. Thus the moment capacity of the splice plates and bolts is $M_s = 63.6 \times 14.32 = 911$ kips-in $= 75.9$ kips-ft. The total moment capacity of this splice with zero compression is thus 75.9 kips-ft, and with 700 kips compression, it is $306 + 75.9 = 382$ kips-ft. The role of compression in providing moment capability is often overlooked in column splice design.

Erection stability. As discussed earlier for base plates, the stability of columns during erection must be a consideration for splice design also. The usual nominal erection load for columns is a 1-kip horizontal force at the column top in any direction. In LRFD format, the 1-kip working load is converted to a factored load by multiplying by a load factor of 1.5. This load of $1 \times 1.5 = 1.5$ kips will require connections that will be similar to those obtained in allowable strength design (ASD) with a working load of 1 kip. It has been established that for major axis bending, the splice is good for 75.9 kips-ft. This means that the 1.5 kip load can be applied at the top of a column $75.9/1.5 = 50.6$ ft tall. Most columns will be shorter than 50.6 ft, but if not, a more robust splice should be considered.

Minor axis stability. If the 1.5-kip erection load is applied in the minor or weak axis direction, the forces at the splice will be as shown in

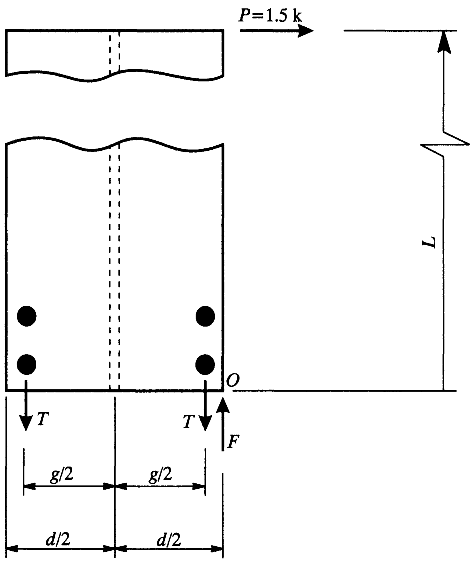


Figure 2.28 Weak-axis stability forces for column splice.

Fig. 2.28. The upper shaft will tend to pivot about point O. Taking moments about point O,

$$PL = T\left(\frac{d}{2} + \frac{g}{2}\right) + T\left(\frac{d}{2} - \frac{g}{2}\right) = Td$$

Thus the erection load P that can be carried by the splice is

$$P = Td/L$$

Note that this erection load capacity (design strength) is independent of the gage g. This is why the AISC splices carry the note, "Gages shown can be modified if necessary to accommodate fittings elsewhere on the column." The standard column gages are $5^1/_2$ and $7^1/_2$ in for beams framing to column flanges. Errors can be avoided by making all column gages the same. The gages used for the column splice can also be $5^1/_2$ or $7^1/_2$ in without affecting erection stability.

If the upper column of Fig. 2.27 is 40 ft long and T is the shear strength of four (two per splice plate) bolts,

$$P = \frac{4 \times 15.9 \times 14.565}{40 \times 12} = 1.93 \text{ kip}$$

Since 1.93 > 1.5, this splice is satisfactory for a 40-ft-long column. If it were not, larger or stronger bolts could be used.

2.2.5.1 Column splices for biaxial bending. The simplest method for designing this type of splice is to establish a flange force (required strength) that is statically equivalent to the applied moments and then to design the bolts, welds, plates, and fillers (if required) for this force.

Major axis bending. If M_x is the major axis applied moment and d is the depth of the deeper of the two columns, the flange force (or required strength) is

$$F_{fx} = \frac{M_x}{d}$$

Minor axis bending. The force distribution is similar to that shown in Fig. 2.28 for erection stability. The force F in the case of actual (factored) design loads can be quite large and will need to be distributed over some finite bearing area as shown in Fig. 2.29. In Fig. 2.29, the bearing area is $2\varepsilon t$, where t is the thickness of the thinner flange, ε is the position of the force F from the toe of the flange of the smaller column, and T is the force per gage line of bolts. The quantities T and F are for each of the two flanges. If M_y is the weak axis applied moment, $M_f = M_y/2$ is the weak axis applied moment per flange. Taking moments about O gives (per flange)

$$M_f = T\left(\frac{b}{2} - \frac{g}{2} - \varepsilon\right) + T\left(\frac{b}{2} + \frac{g}{2} - \varepsilon\right) = T(b - 2\varepsilon)$$

The bearing area is determined by requiring that the bearing stress reaches its design strength at the load F. Thus, $0.75\,(1.8\,F_y)\,(2\varepsilon)\,t = F$, and since from vertical equilibrium $F = 2T$, and

$$0.75(1.8\,F_y)\,t\,\varepsilon = T$$

Thus $M_f = 0.75(1.8F_y)\,t\varepsilon(b - 2\varepsilon)$

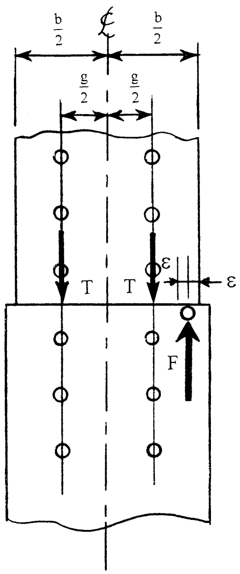


Figure 2.29 Force distribution for minor axis bending.

and solving for ε

$$\varepsilon = \frac{1}{4}b - \frac{1}{2}\sqrt{\left(\frac{b}{2}\right)^2 - \frac{40}{27}\left(\frac{M_f}{F_y}\right)} = \frac{1}{4}b\left[1 - \sqrt{1 - \frac{8}{3}\frac{M_f}{\phi M_{py}}}\right]$$

where $M_{py} = F_y Z_y = \frac{1}{2}F_y t b^2$

This expression for ε is valid as long as

$$M_f \leq \frac{27}{40}\left(\frac{F_y t b^2}{4}\right) = \frac{3}{8}\phi M_{py}$$

When $M_f > 3/8\ \phi M_{py}$, the tension T on the bolts on the bearing side vanishes and Fig. 2.30 applies. In this case, $F = T = 0.75\,(1.8\,F_y)\,t\,(2\varepsilon)$,

$$M_f = T\left(\frac{b+g}{2} - \varepsilon\right)$$

and

$$\varepsilon = \frac{1}{4}(b+g) - \frac{1}{2}\sqrt{\left(\frac{b+g}{2}\right)^2 - \frac{40}{27}\left(\frac{M_f}{F_y t}\right)} = \frac{1}{4}b\gamma\left[1 - \sqrt{1 - \frac{8}{3}\frac{M_f}{\phi M_{py}}\left(\frac{1}{\gamma}\right)^2}\right]$$

where $\gamma = 1 + g/b$

This expression for ε is valid as long as

$$M_f \leq \frac{27 F_y t (b+g)^2}{40 \quad\quad 4} = \frac{3}{8}\gamma^2 \phi M_{py}$$

but T need never exceed Mf/g. The flange force in every case is $F_{fy} = 2T$.

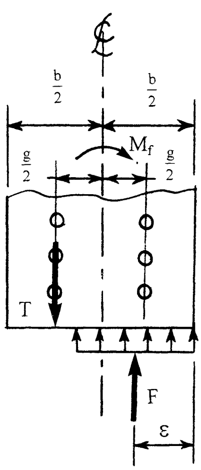


Figure 2.30 Splice force distribution when bolts on bearing side are ineffective.

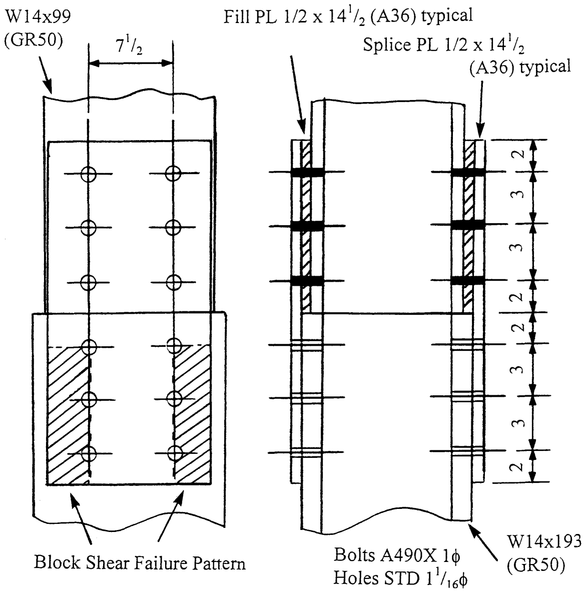


Figure 2.31 Bolted column splice for biaxial bending.

Example: Design a bolted splice for a W14 × 99 upper shaft to a W14 × 193 lower shaft. Design the splice for 15% of the axial capacity of the smaller member plus 20% of the smaller member's bending capacity about either the major or the minor axis, whichever produces the greater flange force F_f. The columns are ASTM A992, the splice plates are ASTM A36, and the bolts are ASTM A490 1-in-diameter X type. The holes are standard 1-1/16-in diameter. The gage is 7-$^1/_2$ in.

The completed splice is shown in Fig. 2.31. The flange force due to tension is

$$F_{f_t} = 0.15 \times \phi \frac{F_y}{2} A_g = \frac{0.15 \times 0.9 \times 50 \times 29.1}{2} = 98.2\,\text{kips}$$

The flange force due to major axis bending is

$$F_{f_x} = \frac{0.20\phi M_{px}}{d} = \frac{0.20 \times 647 \times 12}{14.16} = 110\,\text{kips}$$

The flange force due to minor axis bending is calculated as follows:

$$M_f = \frac{0.20\phi M_{py}}{2} = \frac{0.20 \times 0.9 \times 50 \times 836}{2} = 376\,\text{kips}$$

Check that $M_f = 376 \leq 3/8 \times 0.9 \times 50 \times 83.6 = 1410\,\text{kips-in}$, ok.

Calculate

$$\varepsilon = \frac{1}{4} \times 14.565\left[1 - \sqrt{1 - \frac{376}{1410}}\right] = 0.523\text{ in}$$

Thus, $T = 0.75(1.8 \times 50) \times 0.780 \times 0.523 = 27.5$ kips and $F_{fy} = 2 \times 27.5 = 55.0$ kips.

The flange force for design of the splice is thus

$$f_f = F_{f_t} + \max\{F_{f_x}, F_{f_y}\} = 98.2 + \max\{110, 55.0\} = 208\,\text{kips}$$

Suppose that $M_f > 3/8\phi\, M_{py}$. Let $M_f = 1500$ kips-in, say, $\gamma = 1 + 7.5/14.565 = 1.515$ and check $M_f = 1500$ kips in $< 3/8\ \gamma\ \phi\ M_{py} = (1.515)^2 \times 1410 = 3236$ kips in, so proceeding

$$\varepsilon = \frac{1}{4} \times 14.565 \times 1.515\left[1 - \sqrt{1 - \frac{1500}{3236}}\right] = 1.476\,\text{in}$$

$T = 0.75 \times (1.8 \times 50) \times 0.780 \times 1.476 = 77.5$ kips and $F_{fy} = 2\,T = 155$ kips, which is still less than the maximum possible value of $F_{fy} = 1500/7.5 \times 2 = 400$ kips

Returning to the splice design example, the splice will be designed for a load of 208 kips. Since the columns are of different depths, fill plates will be needed. The theoretical fill thickness is $(15^1/_2 - 14^1/_8)/2 = 11/16$ in, but for ease of erection, the AISC suggests subtracting either $^1/_8$ or $^3/_{16}$ in, whichever results in $^1/_8$-in multiples of fill thickness. Thus, use actual fills $11/16 - 3/16 = 1/2$ in thick. Since this splice is a bearing splice, either the fills must be developed, or the shear strength of the bolts must be reduced. It is usually more economical to do the latter in accordance with AISC Specification Section J6 when the total filler thickness is not more than $^3/_4$ in. Using Section J6, the bolt shear design strength is

$$\phi r_v = 44.2\,[1 - 0.4(0.5 - 0.25)] = 39.8\ \text{kips}$$

The number of bolts required is $208/39.8 = 5.23$ or 6 bolts. The choice of six bolts here may have to be adjusted for bearing/tearout as will be seen later. By contrast, if the fillers were developed, the number of bolts required would be $208\,[1 + 0.5/(0.5 + 0.780)]/44.2 = 6.54$ or 8 bolts. By reducing the bolt shear strength instead of developing the fills, $[(8 - 6)/8]100 = 25\%$ fewer bolts are required for this splice. Next, the splice plates are designed. These plates will be approximately as wide as the narrower column flange. Since the W14 × 99 has a flange width of 14-5/8 in., use a plate 14-1/2 in. wide. The following limit states are checked:

1. Gross area: The required plate thickness based on gross area is $t_p = 208/(0.9 \times 36 \times 14.5) = 0.44$ in. Use a 1/2-in plate so far.
2. Net area: The net area is $A_n = (14.5 - 2 \times 1.125) \times 6.5 = 6.125\ \text{in}^2$, but this cannot exceed 0.85 of the gross area or $0.85 \times 14.5 \times 0.5 = 6.16\ \text{in}^2$.

Since $6.16 > 6.125$, the effective net area $A_e = A_n = 6.125 \text{ in}^2$. The design strength in gross tension is $\phi R_n = 0.75 \times 58 \times 6.125 = 266$ kips > 208 kips, ok.

3. Block shear rupture: Since $b - g < g$, the failure will occur as shown in Fig. 2.31 on the outer parts of the splice plate.

$$A_{gv} = 8 \times 0.5 \times 2 = 8.0 \text{ in}^2$$
$$A_{gt} = (14.5 - 7.5) \times 0.5 = 3.5 \text{ in}^2$$
$$A_{nv} = 8.0 - 2.5 \times 1.125 \times 0.5 \times 2 = 5.1875 \text{ in}^2$$
$$A_{nt} = 3.5 - 1 \times 1.125 \times 0.5 = 2.9375 \text{ in}^2$$
$$F_u A_{nt} = 58 \times 2.9375 = 170 \text{ kips}$$
$$0.6\, F_y A_{gv} = 0.6 \times 36 \times 8.0 = 173 \text{ kips}$$
$$0.6\, F_u A_{nv} = 0.6 \times 58 \times 5.1875 = 181 \text{ kips}$$
$$U_{bs} = 1.0 \text{ (uniform tension)}$$
$$\phi R_{bs} = 0.75\,[1.0 \times 170 + \min\{173, 181\}] = 257 \text{ kips} > 208 \text{ kips, ok}$$

4. Bearing/tearout: Although we have initially determined that six bolts are required, the following bearing/tearout check may require an adjustment in this number:
   a. Bolt shear:
      $\phi R_v = 39.8$ kips
   b. Bearing on splice plate:
      $\phi R_p = 0.75 \times 2.4 \times 1.0 \times 0.5 \times 58 = 52.2$ kips
   c. Bearing on W14 × 99 flange:
      $\phi R_p = 0.75 \times 2.4 \times 1.0 \times .0.780 \times 65 = 91.3$ kips
   d. Tearout on splice plate:
      $\phi R_{to} = 0.75 \times 1.2 \times (2 - 0.5 \times 1.0625) \times 0.5 \times 58 = 38.3$ kips
   e. Tearout on W14 × 99 flange:
      $\phi R_{to} = 0.75 \times 1.2(2 - 0.5 \times 1.0625) \times 0.780 \times 65 = 67.1$ kips

Two more tearout limit states are related to the spacing of the bolts, but these are obviously not critical.

The bearing/tearout limit state is

$$\phi R_{\text{pto}} = 4 \times 39.8 + 2 \times 38.3 = 236 \text{ kips} > 208 \text{ kips, ok}$$

5. Whitmore section:

$$l_w = (6 \tan 30)2 + 7.5 = 14.43 \text{ in}$$
$$\phi R_n = 0.9 \times 36 \times 14.43 \times 0.5 = 234 \text{ kips} > 208 \text{ kips}$$

Note that if $l_w > 14.5$ in, 14.5 in would have been used in the calculation of design strength.

In addition to the checks for the bolts and splice plates, the column sections should also be checked for bearing and block shear rupture. These are not necessary in this case because $t_f = 0.780 > t_p = 0.50$, the edge distances for the column are the same as for the plates, and the column material is stronger than the plate material.

2.2.5.2 Splices in truss chords. These splices must be designed for 50% of the chord load as an axial force, or 2% of the chord load as a transverse force, as discussed in Sec. 5.5.3, even if the load is compression and the members are finished to bear. As discussed earlier, these splices may be positioned in the center of a truss panel and, therefore, must provide some degree of continuity to resist bending. For the tension chord, the splice must be designed to carry the full tensile load.

Example Design the tension chord splice shown in Fig. 2.32. The load is 800 kips (factored). The bolts are A325X, $^7/_8$ in in diameter, $\phi r_v = 27.1$ kips. The load at this location is controlled by the W14 × 90, so the loads should be apportioned to flanges and web based on this member. Thus, the flange load is

$$P_f = \frac{0.710 \times 14.520}{26.5} \times 800 = 311 \text{ kips}$$

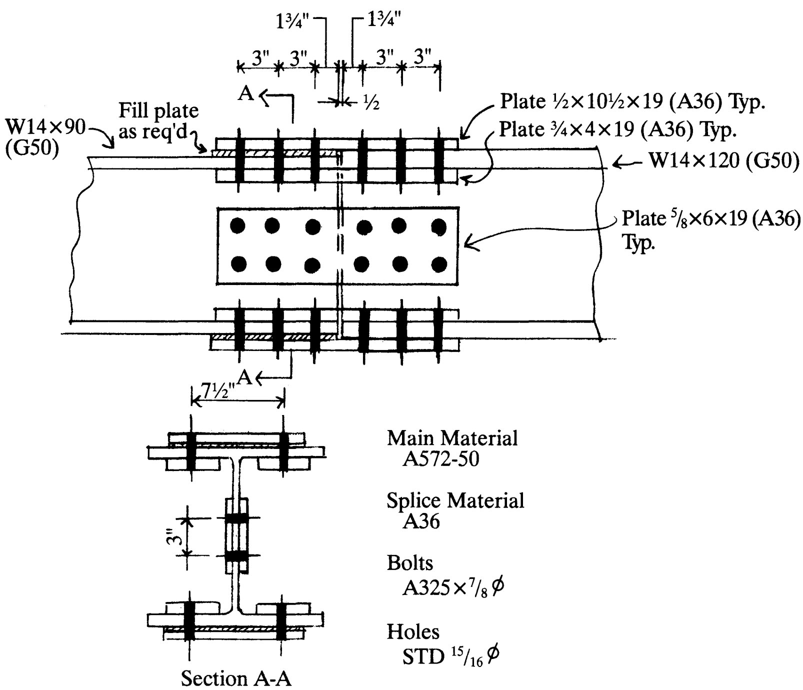


Figure 2.32 Truss chord tension splice.

and the web load is

$$P_w = 800 - 2 \times 311 = 178 \text{ kips}$$

The load path is such that the flange load P_f passes from the W14 × 90 (say) through the bolts into the flange plates and into the W14 × 120 flanges through a second set of bolts. The web load path is similar.

A. Flange connection.

1. Member limit states:

a. *Bolts*: Although not a member limit state, a bolt pattern is required to check the chords. The number of bolts in double shear is 311/(2 × 27.1) = 5.74. Try 6 bolts in 2 rows of 3 as shown in Fig. 2.34. This may need to be adjusted because of bearing/tearout.

b. *Chord net section:* Check to see if the holes in the W14 × 90 reduce its capacity below 800 kips. Assume that there will be two web holes in alignment with the flange holes.

$$A_{\text{net}} = 26.5 + 4 \times 1 \times 0.710 - 2 \times 1 \times .440 = 22.8 \text{ in}^2$$
$$\phi R_{\text{net}} = 22.8 \times 0.75 \times 65 = 1111 \text{ kips} > 800 \text{ kips, ok}$$

c. *Bearing / tearout:* This will be checked after the splice plates are designed.

d. *Block shear fracture:*

$$A_{nv} = (7.75 - 2.5 \times 1.0)0.710 \times 2 = 7.46 \text{ in}^2$$
$$A_{nt} = \left(\frac{(14.520 - 7.5)}{2} - 0.5 \times 1\right)0.710 \times 2 = 4.27 \text{ in}^2$$
$$A_{gv} = 7.75 \times 0.710 \times 2 = 11.0 \text{ in}^2$$
$$A_{gt} = 3.51 \times 0.710 \times 2 = 4.98 \text{ in}^2$$
$$F_u A_{nt} = 65 \times 4.27 = 278 \text{ kips}$$
$$0.6F_y A_{gv} = 0.6 \times 50 \times 11.00 = 330 \text{ kips}$$
$$0.6F_u A_{nv} = 0.6 \times 65 \times 7.46 = 291 \text{ kips}$$
$$U_{bs} = 1.0$$
$$\phi R_{bs} = 0.75[278 + \min\{330, 291\}] = 427 \text{ kips} > 311 \text{ kips, ok}$$

2. Flange plates:

Since the bolts are assumed to be in double shear, the load path is such that one half of the flange load goes into the outer plate, and one half goes into the inner plates.

a. Outer plate:

(1) Gross and net area: Since the bolt gage is $7^1/_2$ in, try a plate 10 $^1/_2$ in wide. The gross area in tension required is

$$A_{gt} = \frac{311/2}{0.9 \times 36} = 4.8\,\text{in}^2$$

and the thickness required is 4.8/10.5 = 0.46 in. Try a plate $^1/_2 \times 10^1/_2$

$$\begin{aligned} A_{gt} &= 0.5 \times 10.5 = 5.25\ \text{in}^2 \\ A_{nt} &= (10.5 - 2 \times 1) \times 0.5 = 4.25\ \text{in}^2 \\ 0.85\,A_{gt} &= 0.85 \times 5.25 = 4.46\ \text{in}^2 \end{aligned}$$

Since 0.85 $A_{gt} > A_{nt}$, use A_{nt} = 4.25 in^2 as the effective net tension area.

$$\phi R_{nt} = 0.75 \times 58 \times 4.25 = 185 \text{ kips} > 311/2 = 156 \text{ kips, ok}$$

Use a plate $^1/_2 \times 10^1/_2$ for the outer flange splice plate for the following limit state checks:

(2) Block shear fracture:

$$\begin{aligned} A_{gv} &= 7.5 \times 0.5 \times 2 = 7.5\ \text{in}^2 \\ A_{gt} &= 1.5 \times 0.5 \times 2 = 1.5\ \text{in}^2 \\ A_{nv} &= (7.5 - 2.5 \times 1)0.5 \times 2 = 5.0\ \text{in}^2 \\ A_{nt} &= (1.5 - .5 \times 1)0.5 \times 2 = 1.0\ \text{in}^2 \\ F_u A_{nt} &= 58 \times 1.0 = 58.0 \text{ kips} \\ 0.6F_y A_{gy} &= 0.6 \times 36 \times 7.5 = 162 \text{ kips} \\ 0.6F_u A_{nv} &= 0.6 \times 58 \times 5.0 = 174 \text{ kips} \\ \phi R_{bs} &= 0.75[58 + \min\{162, 174\}] = 165 \text{ kips} > 156 \text{ kips, ok} \end{aligned}$$

(3) Bearing:

$$\phi R_p = 0.75 \times 2.4 \times 58 \times 0.50 \times .875 \times 6 = 274 \text{ kips} > 156 \text{ kips, ok}$$

Thus, the plate $^1/_2 \times 10^1/_2$ (A36) outer splice plate is ok, but bearing/tearout still needs to be checked.

b. Inner plates:

(1) Gross and net area: The load to each plate is 156/2 = 78 kips. The gross area in tension required is

$$A_{gt} = \frac{78}{0.9 \times 36} = 2.41\ \text{in}^2$$

Try a plate 4 in wide. Then the required thickness is 2.41/4 = 0.6 in. Try a plate ¾ × 4 (A36).

$$A_{gt} = 0.75 \times 4 = 3 \text{ in}^2$$
$$A_{nt} = (4 - 1.0)0.75 = 2.25 \text{ in}^2$$
$$0.85A_{gt} = 0.85 \times 3 = 2.55 \text{ in}^2$$
$$\phi R_{nt} = 0.75 \times 58 \times 2.25 = 97.9 \text{ kips} > 78 \text{ kips, ok}$$

(2) Block shear fracture: Since there is only one line of bolts, this limit state is not possible. The plate will fail in net tension.

The ¾ × 4 (A36) inner splice plates are so far ok, now check bearing/tearout.

3. *Bearing/tearout*: Now that the bolts, the outer plate, and the inner plates have been chosen, bearing/tearout can be checked for the connection as a whole.

a. *Bolt shear:*

$$\phi r_v = 54.2 \text{ kips (double shear)}$$
$$\phi r_v = 27.1 \text{ kips (single shear)}$$

b. *Bearing on W14 × 99 flange:*

$$\phi r_p = 0.75 \times 2.4 \times 0.875 \times 0.710 \times 65 = 72.7 \text{ kips}$$

c. *Bearing on outer plate:*

$$\phi r_p = 0.75 \times 2.4 \times 0.875 \times 0.5 \times 58 = 45.7 \text{ kips}$$

d. *Bearing on inner plate:*

$$\phi r_p = 0.75 \times 2.4 \times 0.875 \times 0.75 \times 58 = 68.5 \text{ kips}$$

e. *Tearout on W14 × 99 flange;* $L_c = 1.75 - 0.5 \times 0.9375 = 1.281$ *in:*

$$\phi r_{to} = 0.75 \times 1.2 \times 1.281 \times 0.710 \times 65 = 53.2 \text{ kips}$$

f. *Tearout on Outer Plate;* $L_c = 1.5 - 0.5 \times 0.9375 = 1.031$ *in:*

$$\phi r_{to} = 0.75 \times 1.2 \times 1.031 \times 0.5 \times 58 = 26.9 \text{ kips}$$

g. *Tearout on Inner Plates;* $L_c = 1.031$ *in:*

$$\phi r_{to} = 0.75 \times 1.2 \times 1.031 \times 0.75 \times 58 = 40.4 \text{ kips}$$

Tearout between bolts will not control in this case since $3 - 0.9375 = 2.0625 > 1.281$ or 1.031.

From the above, the shear/bearing/tearout strength of the flange connection is

$$\begin{aligned}\phi R_{\text{vpt}} &= 2 \times 27.1 \times 2 + 2 \times 26.9 \times 2 + 2 \times 27.1 \times 2 \\ &= 324 \text{ kips} > 311 \text{ kips, ok}\end{aligned}$$

In the expression for ϕR_{vpt}, the first term is for the two bolts in the center, which are controlled by shear; the second term is for the outer two bolts controlled by outer plate edge distance; and the third term is for the two inner bolts again controlled by bolt shear.

This completes the calculation for the flange portion of the splice. The bolts, outer plate, and inner plates, as chosen above, are ok.

B. Web connection:

The calculations for the web connection involve the same limit states as the flange connection, except for chord net section, which involves flanges and web.

1. Member limit states:

a. Bolts: A bolt pattern is required to check the web.

$$\text{Number required} = \frac{178}{2 \times 27.1} = 3.28$$

Try four bolts.

b. Bearing/tearout: This will be checked after the web splice plates are designed.

c. Block shear fracture: Assume the bolts have a 3-in pitch longitudinally.

$$\begin{aligned}A_{nv} &= (4.75 - 1.5 \times 1) \times 0.440 \times 2 = 2.86 \text{ in}^2 \\ A_{nt} &= (3 - 1 \times 1) \times 0.440 = 0.88 \\ A_{gv} &= 4.75 \times 0.440 \times 2 = 4.18 \text{ in}^2 \\ A_{gt} &= 3 \times 0.440 = 1.32 \text{ in}^2 \\ F_u A_{nt} &= 65 \times 0.88 = 57.2 \text{ kips} \\ 0.6F_u A_{nv} &= 0.6 \times 65 \times 2.86 = 112 \text{ kips} \\ 0.6F_y A_{gy} &= 0.6 \times 50 \times 4.18 = 125 \text{ kips} \\ U_{bs} &= 1.0 \\ \phi R_{bs} &= 0.75[57.2 + \min\{115{,}125\}] = 127 \text{ kips} < 178 \text{ kips, no good}\end{aligned}$$

Since the block shear limit state fails, the bolts can be spaced out to increase the capacity. Increase the bolt pitch from the 3 in assumed above to 6 in. Then

$$A_{nv} = (7.75 - 1.5 \times 1) \times 0.440 \times 2 = 5.50 \text{ in}^2$$
$$A_{nt} = 0.88 \text{ in}^2$$
$$A_{gv} = 7.75 \times 0.440 \times 2 = 6.82 \text{ in}^2$$
$$A_{gt} = 1.32 \text{ in}^2$$
$$U_{bs} = 1.0$$
$$F_u A_{nt} = 65 \times 0.88 = 57.2 \text{ kips}$$
$$0.6F_u A_{nv} = 0.6 \times 65 \times 5.50 = 214 \text{ kips}$$
$$0.6F_y A_{gv} = 0.6 \times 50 \times 6.82 = 205 \text{ kips}$$
$$\phi R_{bs} = 0.75[57.2 + \min\{214, 205\}] = 197 \text{ kips} > 178 \text{ kips, ok}$$

The web bolt pattern shown in Fig. 2.32 is the final design. At this point, there are four bolts in the web at 6-in pitch, but the six bolts shown will be required.

2. Web plates: Try two plates, one each side of web, 6 in wide and ½ in thick.

   a. Gross area:

$$\phi R_{gt} = 0.9 \times 36 \times 0.5 \times 6 \times 2 = 194 \text{ kips} > 178 \text{ kips, ok}$$

   b. Net area:

$$A_{nt} = (6 - 2 \times 1) \times 0.5 \times 2 = 4.0 \text{ in}^2$$
$$.85A_{gt} = 0.85 \times 0.5 \times 6 \times 2 = 5.1 \text{ in}^2$$
$$\phi R_{nt} = 0.75 \times 58 \times 4.0 = 174 \text{ kips} < 178 \text{ kips, no good}$$

   Increase web plates to ⅝ in thick. Net area will be ok by inspection.

   c. Block shear rupture: This is checked as shown in previous calculations. It is not critical here.

3. *Bearing/tearout:* The bolt pattern and plates are now known, so this combined limit state can be checked.

   a. Bolt shear:

$$\phi r_v = 54.2 \text{ kips (double shear)}$$
$$\phi r_v = 27.1 \text{ kips (single shear)}$$

   b. Bearing on W14 × 99 web:

$$\phi r_p = 0.75 \times 2.4 \times 0.875 \times 0.440 \times 65 = 45.0 \text{ kips}$$

c. Bearing on splice plates:

$$\phi r_p = 0.75 \times 2.4 \times 0.875 \times 0.625 \times 2 \times 58 = 114 \text{ kips}$$

d. Tearout on W14 × 99 web:

$$\begin{aligned} L_c &= 1.75 - 0.5 \times .9375 = 1.281 \\ \phi r_{to} &= 0.75 \times 1.2 \times 1.281 \times 0.440 \times 65 = 33.0 \text{ kips} \end{aligned}$$

e. Tearout on splice plates:

$$\phi r_{to} = 0.75 \times 1.2 \times 1.281 \times 0.625 \times 2 \times 58 = 83.6 \text{ kips}$$

Tearout between bolts will not control in this case.

From the above, the shear/bearing/tearout strength of the connection is

$$\phi R_{\text{vpt}} = 2 \times 33.0 + 2 \times 45.0 = 156 \text{ kips} < 178 \text{ kips, no good}$$

Add two bolts in the web. The 6-in pitch become 3-in pitch as shown in Fig. 2.32. The shear/bearing/tearout capacities per bolt given above do not change. Tearout between bolts is still not critical. Thus

$$\phi R_{\text{vpt}} = 2 \times 33.0 + 4 \times 45.0 = 246 \text{ kips} > 178 \text{ kips, ok}$$

Note that, in this case, none of the bolts was able to achieve its double shear value.

4. Additional checks because of change in web bolts pattern:

a. Block shear fracture:

$$\begin{aligned} A_{nv} &= (7.75 - 2.5 \times 1.0)\, 0.440 \times 2 = 4.62 \text{ in}^2 \\ A_{gv} &= 7.75 \times 0.440 \times 2 = 6.82 \text{ in}^2 \\ A_{nt} &= (3 - 1 \times 1.0) \times 0.440 = 0.88 \text{ in}^2 \\ F_u A_{nt} &= 65 \times 0.88 = 57.2 \text{ kips} \\ 0.6 F_u A_{nv} &= 0.6 \times 65 \times 4.62 = 180 \text{ kips} \\ 0.6 F_y A_{gv} &= 0.6 \times 50 \times 6.82 = 205 \text{ kips} \\ U_{bs} &= 1.0 \\ \phi R_{bs} &= 0.75\, [57.2 + \min\{180, 205]\} = 178 \text{ kips} = 178 \text{ kips, ok} \end{aligned}$$

No other design check must be done. The final design is shown in Fig. 2.32.

If this were a non-bearing compression splice, the splice plates would be checked for buckling. The following paragraph shows the method, which is not required for a tension splice.

5. Buckling:

The plates at the web splice line of 4-in length can be checked against a load of 178/2 = 89 kips/plate. The slenderness ratio is

$$\frac{Kl}{r} = 0.65 \times 4.0 \times \sqrt{\frac{12}{0.0625}} = 14.4$$

Since this is less than 25, AISC Specification Section J4.4 allows the plate to be checked for yield rather than buckling. This has already been done.

This limit state is checked for the flange plates also.

2.3 Moment Connections

2.3.1 Introduction

The most commonly used moment connection is the field-welded moment connection as shown in Fig. 2.33*a*. This connection is in common use in all regions of the United States, where the seismic design category

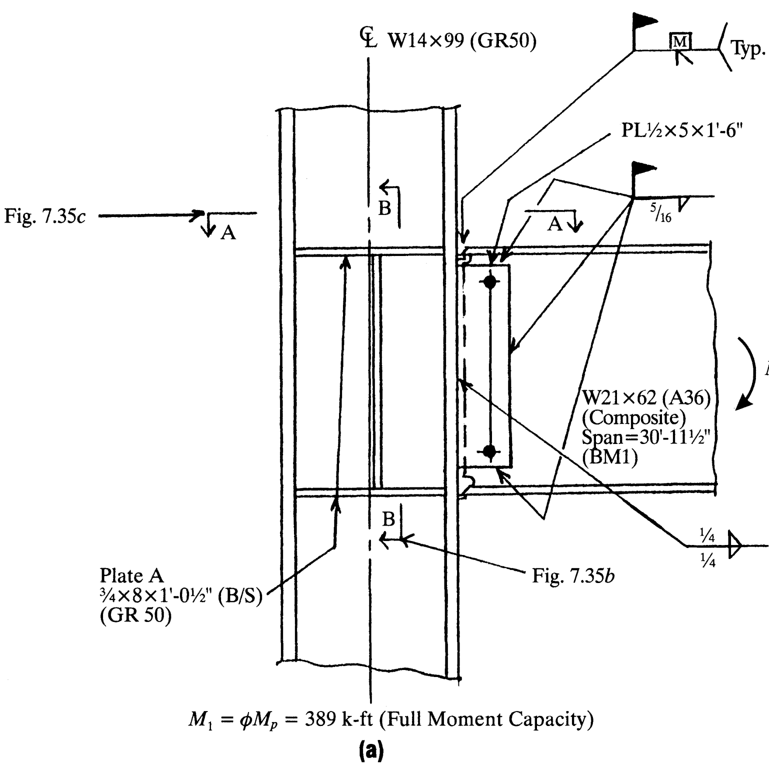


Figure 2.33*a* Field-welded moment connection.

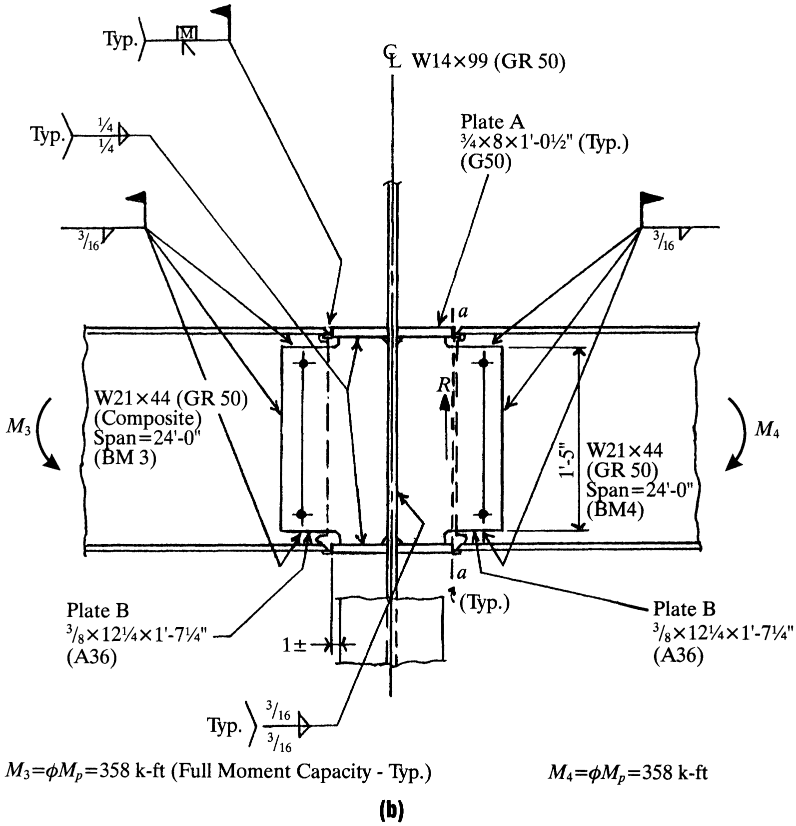
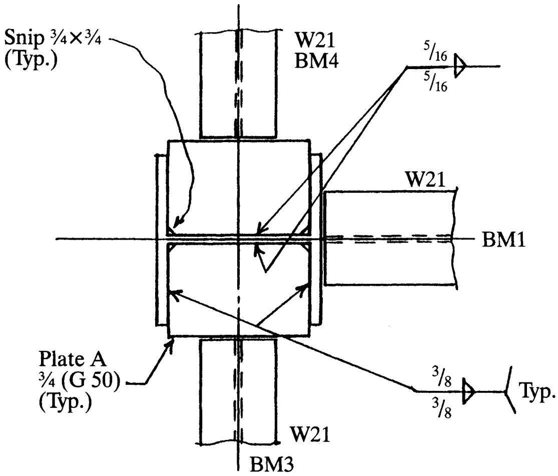


Figure 2.33*b* Section B-B of Fig. 2.33*a*.

(SDC) is A, B, or C, and the response modification factor R is three or less (AISC Seismic Provisions 2005).

2.3.2 Example—three-way moment connection

The moment connection of Fig. 2.33*a* is a three-way moment connection. Additional views are shown in Figs. 2.33*b* and 2.33*c*. If the strong axis connection requires stiffeners, there will be an interaction between the flange forces of the strong and weak axis beams. If the primary function of these moment connections is to resist lateral maximum load from wind or seismic sources, the interaction can generally be ignored because the maximum lateral loads will act in only one direction at any one time. If the moment connections are primarily used to carry gravity loads, such as would be the case when stiff floors with small deflections and high natural frequencies are desired, there will be interaction between the weak and strong beam flange forces. The calculations here



(c)

Figure 2.33*c* Section A-A of Fig. 2.33*a*.

will be for both a wind or a seismic condition in a region of low to moderate seismicity (SDC A, B, or C, and R = 3), and gravity condition. Thus, interaction will be included.

The load path through this connection that is usually assumed is that the moment is carried entirely by the flanges, and the shear entirely by the web. This load path has been verified by testing (Huang et al., 1973) and will be the approach used here. Proceeding to the connection design, the strong axis beam, beam no. 1, will be designed first.

Beam no. 1 W21 × 62 (A36) composite. The flange connection is a full penetration (referred to as a CJP weld in AWS D1.1) weld, so no design is required. The column must be checked for stiffeners and doublers.

Stiffeners. The connection is to be designed for a given moment of ϕM_b = 389 kips-ft. The given beam moment of ϕM_b = 389 kips-ft can only be achieved if the column is strong enough to support it. The full plastic moment capacity of the column is

$$\phi M_p = 0.9 \times \frac{50}{12} \times 173 = 649\,\text{kips-ft}$$

Thus, since $\phi M_b = 389 < 2 \times 649 = 1300$, the column can support the specified beam moment.

Thus, the flange force F_f is

$$F_f = \frac{\phi M_b}{d - t_f} = \frac{389 \times 12}{(20.99 - 0.615)} = 229\text{ kips}$$

From Table 4-1 of the AISC 13th Edition *Manual of Steel Construction*.

Web yielding: $P_{wy} = P_{wo} + t_b P_{wi} = 167 + 0.615 \times 24.3 = 182$ kips $<$ 229 kips, thus stiffeners are required at both flanges.

Web buckling: $P_{wb} = 260$ kips > 229 kips − no stiffener required at compression flange.

Flange bending: $P_{fb} = 171$ kips $<$ 229 kips − stiffener required at tension flange.

From the preceding three checks (limit states), a stiffener is required at both flanges. For the tension flange, the total stiffener force is 229 − 171 = 58 kips and for the compression flange, the stiffener force is 229 − 189 = 40 kips. But the loads may reverse, so use the larger of 58 and 40 as the stiffener force for both flanges. Then, the force in each stiffener is $^{58}/_{2} = 29$ kips, both top and bottom.

Determination of stiffener size. The minimum stiffener width w_s is

$$\frac{b_{fb}}{3} - \frac{t_{wc}}{2} = \frac{8.24}{3} - \frac{0.485}{2} = 2.5 \text{ in}$$

Use a stiffener $6^1/_2$ in wide to match column.

The minimum stiffener thickness t_s is

$$\frac{t_{fb}}{2} = \frac{0.615}{2} = 0.31 \text{ in}$$

Use a stiffener at least $^3/_8$ in thick.

The minimum stiffener length l_s is

$$\frac{d_c}{2} - t_{fc} = \frac{14.2}{2} - 0.78 = 6.3 \text{ in}$$

The minimum length is for a "half depth" stiffener, which is not possible in this example because of the weak axis connections. Therefore, use a full-depth stiffener of $12^1/_2$ in length.

A final stiffener size check is a plate buckling check that requires that

$$t_s \geq \frac{Ws}{15} = \frac{6.5}{15} = 0.433 \text{ in}$$

Therefore, the minimum stiffener thickness is $^1/_2$ in. The final stiffener size for the strong axis beam is $^1/_2 \times 6^1/_2 \times 12^1/_2$. The contact area of this stiffener against the inside of the column flange is 6.5 − 0.75 = 5.75 due to the snip to clear the column web to flange fillet. The stiffener design strength is thus $0.9 \times 36 \times 5.75 \times 0.5 = 93.2$ kips > 29 kips, ok.

Welds of stiffeners to column flange and web. Putting aside for the moment that the weak axis moment connections still need to be considered and will affect both the strong axis connection stiffeners and welds, the welds for the $^1/_2 \times 6^1/_2 \times 12^1/_2$ strong axis stiffener are designed as follows. For the weld to the inside of the flange, the force to be developed by the weld to the connected portion is 29 kips. Thus, the $5^3/_4$ contact, which is the connected portion, is designed for 29 kips. The weld to the flange is thus

$$D_f = \frac{29}{2 \times 5.75 \times 1.392 \times 1.5} = 1.21$$

An AISC minimum fillet weld is indicated. The factor 1.5 in the denominator above comes from the AISC Specification, Section J2.4, for transversely loaded fillets. The weld to the web has a length $12.5 - 0.75 - 0.75 = 11.0$, and is designed to transfer the unbalanced force in the stiffener to the web. The unbalanced force in the stiffener is 29 kips in this case. Since the weld at the web and the weld at the flange do not share load in this case, both the longitudinally and transversely loaded welds can develop their full strength. Thus,

$$D_w = \frac{29}{2 \times 11.0 \times 1.392} = 0.95$$

An AISC minimum fillet is indicated.

2.3.2.1 Doublers. The beam flange force (required strength) delivered to the column is $F_f = 229$ kips. The design shear strength of the column $\phi V_v = 0.9 \times 0.6 \times 50 \times 0.485 \times 14.16 = 185$ kips < 229 kips, so a doubler appears to be required. However, if the moment that is causing doublers is $\phi M_b = 389$ kips-ft, then from Fig. 2.34, the column story shear is

$$V_s = \frac{\phi M_b}{H}$$

where H is the story height. If $H = 13$ ft,

$$V_s = \frac{389}{13} = 30\,\text{kips}$$

and the shear delivered to the column web is $F_f - V_s = 229 - 30 = 199$ kips. Since 199 kips > 185 kips, a doubler (or doublers) is still indicated. If some panel zone deformation is acceptable, the AISC Specification Section J10.6, Formula J10-11 or J10-12, contains an extra term which increases the panel zone strength. The term is

$$\frac{3b_{fc}t_{fc}^2}{d_b d_c t_{wc}} = \frac{3 \times 14.6 \times 0.780^2}{21.0 \times 14.2 \times 0.485} = 0.184$$

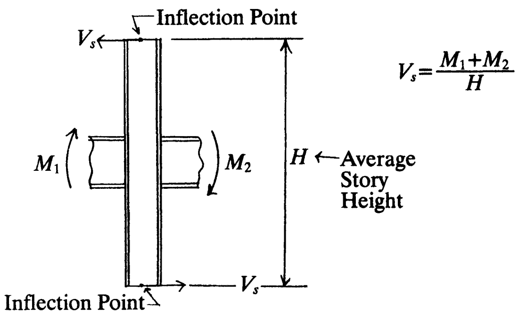


Figure 2.34 Relationship between column story shear and the moments which induce it.

and if the column load is less than 0.75 $P_y = 0.75 \times A_c F_{yc} = 0.75 \times 29.1 \times 50 = 1091$ kips, which is the usual case,

$$\phi V_v = 185 \times 1.184 = 219 \text{ kips}$$

Since 219 kips > 199 kips, no doubler is required. In a high-rise building where the moment connections are used for drift control, the extra term can still be used, but an analysis that includes inelastic joint shear deformation should be considered.

Placement of doubler plates. If a doubler plate or plates is/are required in this example, the most inexpensive arrangement is to place the doubler plate against the column web between the stiffeners (the panel zone) and to attach the weak axis shear connection plates, plates B, to the face of the doubler. This is permissible provided that the doubler is capable of carrying the entire weak axis shear load $R = 163$ kips on one vertical cross section of the doubler plate. To see this, consider Fig. 2.35.

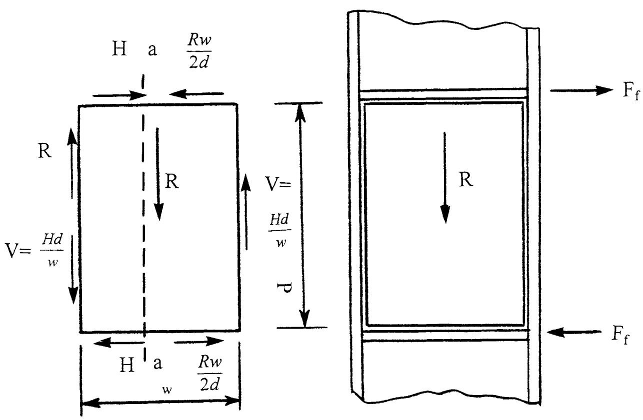


Figure 2.35 Equilibrium of doubler plate with weak axis shear load.

The portion of the shear force induced in the doubler plate by the moment connection flange force F_f is H. For the doubler to be in equilibrium under the forces H, vertical shear forces $V = Hd/w$ must exist. The welds of the doubler at its four edges develop the shear strength of the doubler. Let the shear force R from the weak axis connection be applied to the face of the doubler at or near its horizontal center as shown in Fig. 2.35. If it is required that all of the shear R can be carried by one vertical section a-a of Fig. 2.35, that is, $1.0 \times 0.6 \times \mathrm{F_y t_d d} \geq$ R, where $\mathrm{t_d}$ is the doubler thickness and F_y is the yield strength of the doubler (and the column), then the free-body diagram of Fig. 2.35 is possible. In this figure, all of the shear force R is delivered to the side of the doubler where it is opposite in direction to the shear delivered by the moment connection, thereby avoiding over-stressing the other side where the two shears would add. Since the doubler and its welds are capable of carrying V or R alone, they are capable of carrying their difference. The same argument applies to the top and bottom edges of the doubler. Also, the same argument holds if the moment and/or weak axis shear reverse(s). The validity of this approach is based on the lower bound theorem of limit analysis.

2.3.2.2 Associated shear connections—Beam 1. The specified shear for the web connection is $R = 163$ kips, which is the shear capacity of the W21 × 62 (A36) beam. The connection is a shear plate with two erection holes for erection bolts. The shear plate is shop welded to the column flange and field welded to the beam web. The limit states are plate gross shear, weld strength, and beam web strength.

Plate gross shear. Try a plate $^1/_2 \times 18$

$$\phi R_{gv} = 0.5 \times 18 \times 0.9 \times 0.6 \times 36 = 175 \text{ kips} > 163 \text{ kips, ok}$$

Plate net shear need not be checked here because it is not a valid limit state.

Weld-to-column flange. This weld sees shear only. Thus

$$D = \frac{163}{2 \times 18 \times 1.392} = 3.25; \text{ use } {}^1/_4 \text{ FW}$$

Weld-to-beam web. This weld sees the shear plus a small couple. Using AISC 13th Edition Manual Table 8-8, $l = 18$, $kl = 4.25$, $k = 0.24$, $x = 0.04$, $xl = 0.72$, $al = 4.28$, $a = 0.24$, $c = 2.71$, and

$$D = \frac{163}{0.75 \times 2.71 \times 18} = 4.46$$

Thus a $^5/_{16}$ fillet weld is satisfactory.

Beam web. To support a $^5/_{16}$ fillet weld on both sides of a plate, AISC LRFD Manual Table 10.2 shows that a 0.476-in web is required. For a $^5/_{16}$ fillet on one side, a 0.238-in web is required. Since the W21 × 62 web is 0.400 in thick, it is ok.

Beam nos. 3 and 4 W21 × 44 (G50) composite. The flange connection is a full penetration weld, so again, no design is required. Section A-A of Fig. 2.33a shows the arrangement in plan. See Fig. 2.33*c*. The connection plates A are made $^1/_4$ in thicker than the W21 × 44 beam flange to accommodate under and over rolling and other minor misfits. Also, the plates are extended beyond the toes of the column flanges by $^3/_4$ to 1 in to improve ductility. The plates A should also be welded to the column web, even if not required to carry load, to provide improved ductility. A good discussion of this is contained in the AISC 13th Edition *Manual of Steel Construction*, pp. 12-14 through 12-19.

The flange force for the W21 × 44 is based on the full moment capacity as required in this example, so $\phi M_p = 358$ kips-ft. For gravity moments, the beam moments counteract each other, and the column bending strength is not an issue. For lateral moments, however, the beam moments add, and the column strength may limit the beam moments. The weak-axis column design strength is

$$\phi M_p = 0.9 \times \frac{50}{12} \times 83.6 = 314 \text{ kips-ft}$$

Therefore, for lateral loads, the beam plastic moment cannot be achieved because $2 \times 358 > 2 \times 314$.

For lateral loads, the maximum beam moment is $\phi M_b = 314$ kips-ft.

In summary, for gravity loads, $\phi M_b = \phi M_p = 358$ kips-ft and the flange force is

$$F_f = \frac{358 \times 12}{(20.7 - 0.45)} = 212 \text{ kips}$$

and for lateral loads, $\phi M_b = 314$ kips-ft and the flange force is

$$F_f = \frac{314 \times 12}{(20.7 - 0.450)} = 186 \text{ kips}$$

Figure 2.36 shows the distribution of forces on the plates A, including the forces from the strong axis connection. The weak axis gravity force of 212 kips is distributed one-fourth to each flange and one-half to the web. This is done to cover the case when full gravity loads are not present on each side. In this case, all of the 212 kips must be passed to the flanges. To see this, imagine that beam 4 is removed and the plate A for beam 4 remains as a back-up stiffener. One half of the 212 kips from beam 3 passes into the beam 3 near side column flanges, while the other half is passed through

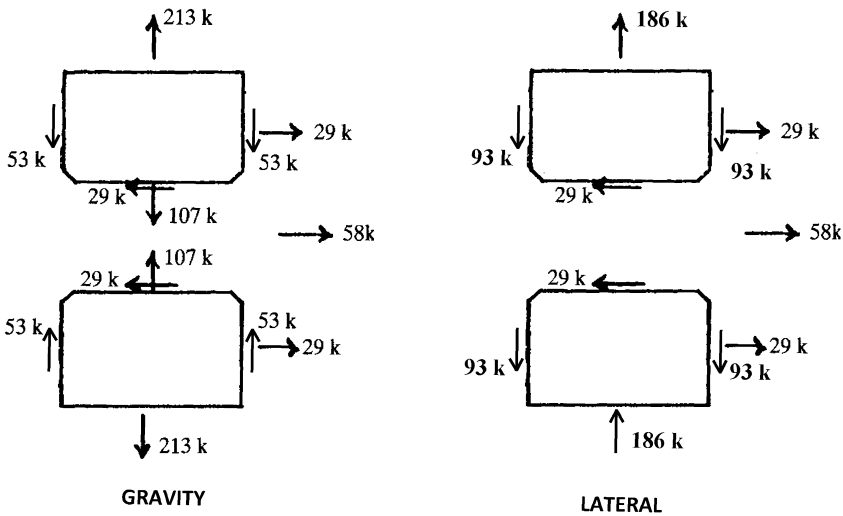


Figure 2.36 Distribution of forces on plates A.

the column web to the back-up stiffener, and thence into the far side flanges, so that all of the load is passed to the flanges. This is the load path usually assumed for gravity loads, although others are possible.

The weak-axis lateral load is distributed one-half to each flange and none to the web. As in the unbalanced gravity load case, all load must be delivered to the flanges. Although no load goes to the web, the stiffener would still be welded to the web for ductility purposes.

Merging of stiffeners from strong and weak axis beams. The strong axis beam, beam no. 1, required stiffeners $^1/_2 \times 6^1/_2 \times 12^1/_2$. The weak axis beams no. 3 and no. 4 require plates A $^3/_4 \times 8 \times 12^1/_2$. These plates occupy the same space because the beams are all of the same depth. Therefore, the larger of the two plates is used, as shown in Fig. 2.33*c*.

Since the stiffeners are merged, the welds that were earlier determined for the strong axis beam must be revisited.

Weld to web. From the worst case of Fig. 2.36,

$$D_w = \frac{\sqrt{29^2 + 107^2}}{2 \times 11.0 \times 1.392} = 3.59$$

Use a $^1/_4$ fillet weld or AISC minimum.

Weld to flanges. From the worst case of Fig. 2.36,

$$D_f = \frac{\sqrt{29^2 + 93^2}}{2 \times 6.25 \times 1.392} = 5.60$$

This indicates a $^3/_8$ fillet weld is required.

In the above weld size calculations, the worst case of gravity loads and lateral loads is used. If it is known that one or the other only exists, only that cases need be considered. When it is not known whether the loads are gravity or lateral, the worst case presumed here must be used.

Note also, that is the weld size calculations, the AISC Specification Section J2.4, which allows for increased strength of obliquely loaded fillet welds, is not used. The compatibility requirements associated with obliquely loaded fillets of different sizes in the same group are complex and are not considered here.

Stresses in stiffeners (plate A). The weak axis beams are G50 steel and are butt welded to plates A. Therefore, plates A should also be G50 steel. Previous calculations involving this plate assumed it was A36, but changing to G50 will not change the final results in this case because the stiffener contact force is limited by the beam no. 1 delivered force rather than the stiffener strength.

The stiffener stresses for the flange welds are, from Fig. 2.36 (worst case),

$$f_v = \frac{93}{0.75 \times 6.25} = 19.8\,\text{ksi} < 1.0 \times 0.6 \times 50 = 30\,\text{ksi, ok}$$

$$f_a = \frac{29}{0.75 \times 6.25} = 6.19\,\text{ksi} < 0.9 \times 50 = 45\ \text{ksi, ok}$$

and for the web welds

$$f_v = \frac{29}{0.75 \times 11} = 3.5\,\text{ksi} < 30\ \text{ksi, ok}$$

$$f_a = \frac{107}{0.75 \times 11} = 13.0\ \text{ksi} < 45\ \text{ksi, ok}$$

2.3.2.3 Associated shear connections—beams 3 and 4. The specified shear for these beams is $R = 107$ kips.

Weld-to-beam web. As with the strong axis beam web connection, this is a field-welded connection with bolts used for erection only. The design load (required strength) is $R = 107$ kips. The beam web shear R is essentially constant in the area of the connection and is assumed to act at the edge of plate A (Section a-a of Fig. 2.33*b*). This being the case, there will be a small eccentricity on the C-shaped field weld. Following AISC 13th Edition Manual Table 8-8, $l = 17$, $kl = 4$, $k = 0.24$, $x = 0.04$, $xl = 0.68$, $al = 4.25 - 0.68 = 3.57$, and $a = 0.21$. From Table 8-8 by interpolation, $c = 2.80$, and the weld size required is

$$D = \frac{107}{0.75 \times 2.80 \times 17} = 3.00$$

which indicates that a $^3/_{16}$ fillet weld is required.

Plate B (shear plate) gross shear. Try a $^3/_8$ plate of A36 steel. Then

$$\phi R_v = 1.0 \times 0.6 \times 36 \times 0.375 \times 17 = 138 \text{ kips} > 107 \text{ kips, ok}$$

Weld of plate B to column web. This weld carries all of the beam shear R = 107 kips. The length of this weld is 17.75 in. Thus

$$D = \frac{107}{2 \times 17.75 \times 1.392} = 2.17$$

A $^3/_{16}$ fillet weld is indicated. Because this weld occurs on both sides of the column web, the column web thickness should satisfy the relationship $0.75 \times 0.6 \times 65t_w \geq 1.392 \times D \times 2$ or $t_w > 0.207$. Since the column web thickness is 0.485 in, the web can support the $^3/_{16}$ fillets. The same result can be achieved using AISC LRFD Manual Table 10.2.

Weld of plate B to plates A. There is a shear flow $q = VQ/I$ acting on this interface, where $V = R = 107$ kips, Q is the statical moment of plate A with respect to the neutral axis of the I section formed by plates A as flanges and plate B as web. Thus

$$I = \frac{1}{12} \times 0.375 \times 19.25^3 + 0.75 \times 12.5 \times \left(\frac{19.25 + 0.75}{2}\right)^2 \times 2$$
$$= 2100 \text{ in}^4$$
$$Q = 0.75 \times 12.5 \times 10 = 93.8 \text{ in}^3$$

and

$$q = \frac{107 \times 93.8}{2100} = 4.78 \text{ kips/in}$$

Thus,

$$D = \frac{4.78}{2 \times 1.392} = 1.72$$

Since plate A is $^3/_4$ in thick, the AISC minimum fillet weld is $^1/_4$ in.

The total shear flow force acting on plate A is $4.78 \times 6.25 = 29.9$ kips. This force does not affect the welds of stiffener A to the column. Rather, stiffener A can be considered an extension of the beam flange, and the shear flow force is taken as part of the flange force. Since the beam flange is full penetration welded to the stiffener A, no further analysis is required.

2.4 Shear Connections

2.4.1 Introduction

Shear connections are the most common type of connections on every job. They are generally considered to be "simple" connections in that the beams supported by them are "simple" beams, that is, no bending moment at the beam ends. There are two basic types of shear connections, framed and seated.

2.4.2 Framed connections

These are the familiar double-angle, single-angle, single-shear plate, and shear end-plate connections. They are called *framed connections* because they connect beams, web-to-web, directly. Figure 2.37*a* shows a typical double-angle connection and Fig. 2.37*b* shows a shear end-plate connection. These and other types of framed connections can be easily designed using the design aids (charts, tables) contained in the AISC *Manual of Steel Construction*. A shear end-plate and single plate shear connection will be designed in detail in the next two examples. The other types are designed in a similar manner.

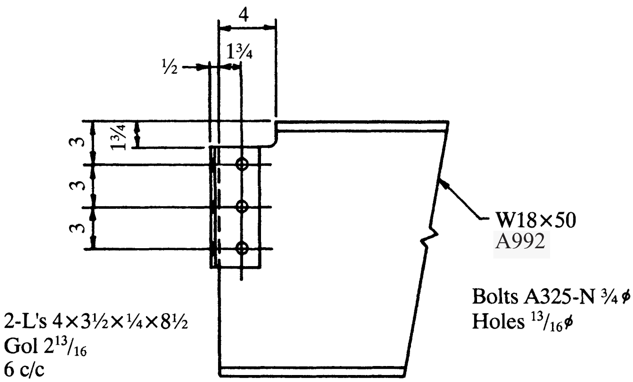


Figure 2.37*a* Double-angle framed connection.

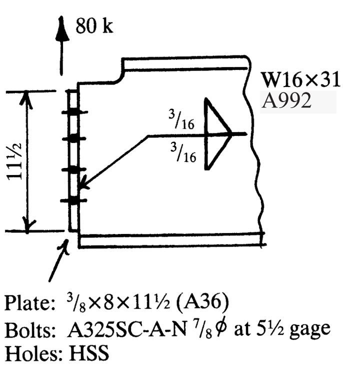


Figure 2.37*b* Shear end-plate connection.

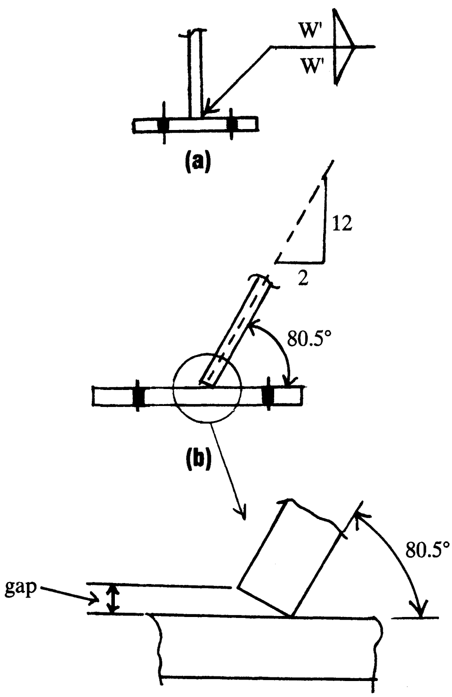


Figure 2.38 Geometry of skewed joint.

Example—shear end plate design. One of the principal uses of shear end-plate connections is for skewed connections. Suppose the W16 beam of Fig. 2.37*b* is skewed $9\frac{1}{2}°$ (a 2 on 12 bevel) from the supporting beam or column as shown in Fig. 2.38. The nominal weld size is that determined from the analysis with the plate perpendicular to the beam web (Fig. 2.38a). This is denoted W', where $W' = {}^{2.6}/_{16} = 0.1625$. The effective throat for this weld is $t_e = 0.7071$ $W' = 0.707(0.1625) = 0.115$ in. If the beam web is cut square, the gap on the obtuse side is $0.275\sin(9.5) < {}^{1}/_{16}$, so it can be ignored.

The weld size, W, for a skewed weld is

$$W = t_e\left(\frac{2\sin\Phi}{2}\right) + g$$

where Φ is the dihedral angle.

For the obtuse side, $\Phi = 90 + 9.5 = 99.5$,

$$W = 0.115\left(\frac{2\sin(99.5)}{2}\right) + 0 = 0.1755;\ \frac{3}{16}\ \text{fillet weld}$$

For the acute side, $\Phi = 90 - 9.5 = 80.5$,

$$W = 0.115\left(\frac{2\sin(80.5)}{2}\right) + 0 = 0.1486;\ \frac{3}{16}\ \text{fillet weld}$$

In this case, the fillet sizes remain the same as the orthogonal case. In general, the obtuse side weld will increase and the acute side weld will decrease, as will be seen in the next section.

2.4.3 Skewed connections

The shear end-plate example of the previous section ended with the calculation of welds for a skewed connection. There are many types of skewed connections. The design recommendations for economy and safety have been reviewed by Kloiber and Thornton (1997). This section is largely taken from that paper.

Skewed connections to beams. The preferred skewed connections for economy and safety are single plates (Fig. 2.39) and end plates (Fig. 2.40). Single bent plates (Fig. 2.41) and eccentric end plates also work well at very acute angles. The old traditional double bent plate connections are

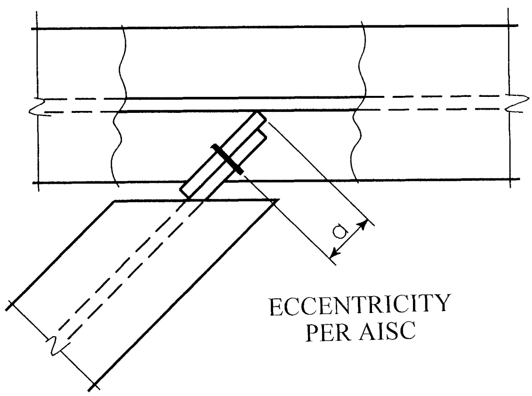


Figure 2.39 Shear tab (single plate). (*Courtesy of Kloiber and Thornton, with permission from ASCE.*)

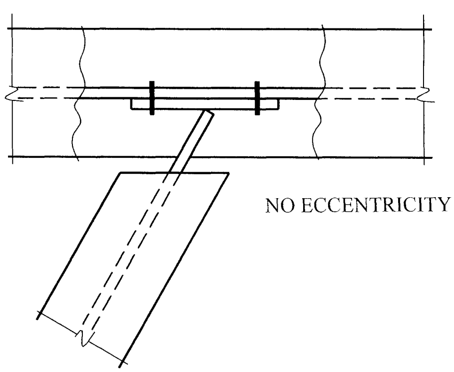
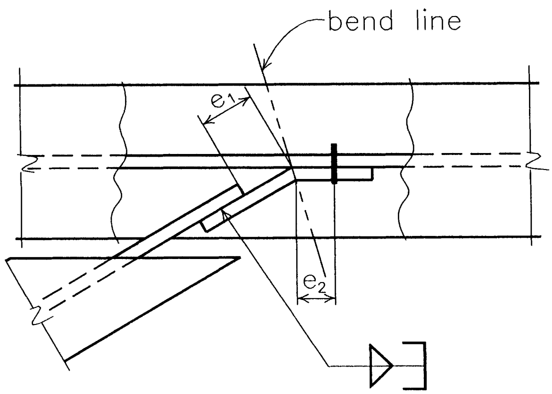


Figure 2.40 Shear end plate. (*Courtesy of Kloiber and Thornton, with permission from ASCE.*)



e_1 and e_2 are connection eccentricities
THERE ARE NO MEMBER ECCENTRICITIES

Figure 2.41 Bent plate. (*Courtesy of Kloiber and Thornton, with permission from ASCE.*)

difficult to accurately fit and are expensive to fabricate. There are also quality (safety) problems with plate cracking at the bend line as the angle becomes more acute.

Single plates (Fig. 2.39) are the most versatile and economical skewed connection with excellent dimensional control when using short slotted holes. While capacity is limited, this is usually not a problem because skewed members generally carry smaller tributary area. Single plates can be utilized for intersection angles of 90° to 30°. Traditionally, snug-tight bolts were preferred because they were more economical and greatly simplified installation. However, the advantages of TC bolt installation often make it more economical to pretension the bolts, though, since the bolts are not required to be pretensioned, no preinstallation verification is required for these connections. Leaving the bolts snug-tight can eliminate the "banging bolt" problem, which occurs in single plate connections when pretensioned bolts slip into bearing. There are AISC 13th Edition Manual (2005) tables available, which can be used to select the required plate size and bolts along with the weld capacity for the required load. This connection has an eccentricity related to the parameter, a, of Fig. 2.39. The actual eccentricity depends on support rigidity, hole type, and bolt installation. The actual weld detail, however, has to be developed for the joint geometry. Welding details for skewed joints were discussed in Sec. 1.3.7.

End plates (Fig. 2.40) designed for shear only are able to provide more capacity than single plates and if horizontal slots are utilized with shug-tight bolts in bearing some dimension adjustment is possible. Holes gages can be adjusted to provide bolt access for more acute skews. A constructability problem can arise when there are opposing beams that

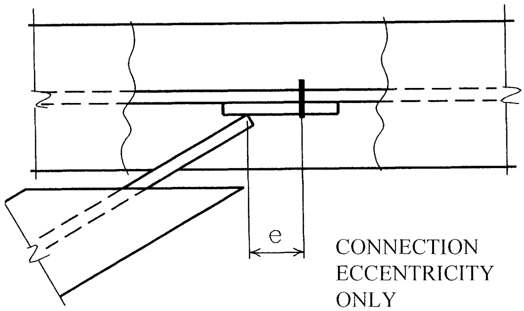


Figure 2.42 Eccentric end plate. (*Courtesy of Kloiber and Thornton, with permission from ASCE.*)

limit access to the back side of the connection. These end-plate connections can be sized using the AISC (2005) tables to select plate size, bolts, and weld capacity. Note that there is no eccentricity with this joint. The weld detail, however, has to be adjusted for the actual geometry of the joint in a manner similar to the shear plate.

Single bent plates as in Fig. 2.41 can be sized for either welded connections using the procedures in the AISC *Manual of Steel Construction* for single angle connections. These involve two eccentricities, e_1 and e_2, from the bend line.

Eccentric end plates (Fig. 2.42) can be easily sized for the eccentricity, e, using the tables in the AISC *Manual of Steel Construction* for eccentric bolt groups.

Skewed connections to columns. Skewed connections to wide-flange columns present special problems. Connections to webs have very limited access, and except for columns where the flange width is less than the depth, or for skews less than 30°, connections to flanges are preferred.

When connecting to column webs, it may be possible to use either a standard end plate or eccentric end plate as shown in Figs. 2.43 and 2.44.

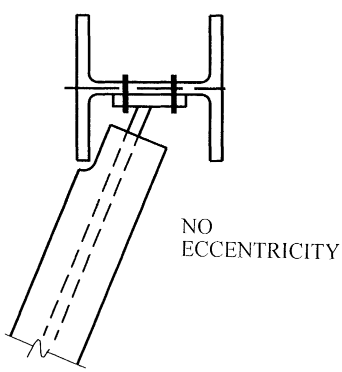


Figure 2.43 End plate. (*Courtesy of Kloiber and Thornton, with permission from ASCE.*)

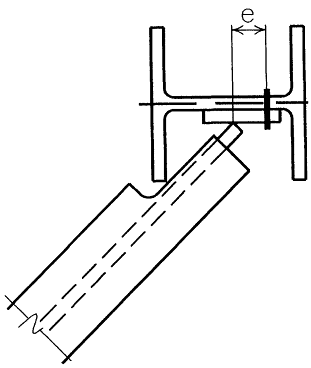


Figure 2.44 Eccentric end plate. (*Courtesy of Kloiber and Thornton, with permission from ASCE.*)

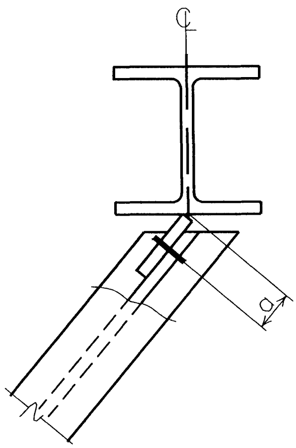


Figure 2.45 Single plate (extended shear tab). (*Courtesy of Kloiber and Thornton, with permission from ASCE.*)

Single-plate connections should not be used unless the bolts are positioned outside the column flanges. In such cases, the connection should be checked as an extended shear tab as outlined later in this chapter.

Skewed connections to column flanges will also be eccentric when the beam is aligned to the column centerline. However, if the beam alignment is centered on the flange, as shown in Fig. 2.45, the minor axis eccentricity is eliminated and the major axis eccentricity will not generally govern the column design. The connection eccentricity is related to the parameter, a, here in the same way as was discussed for Fig. 2.39.

When the beam is aligned to the column centerline, single plates (Fig. 2.46), eccentric end plates (Fig. 2.47 and 2.48), or single bent plates (Fig. 2.49) can be used. The eccentricity for each of these connections is again similar to that for the same connection to a beam web. An additional eccentricity, e_y, which causes a moment about the

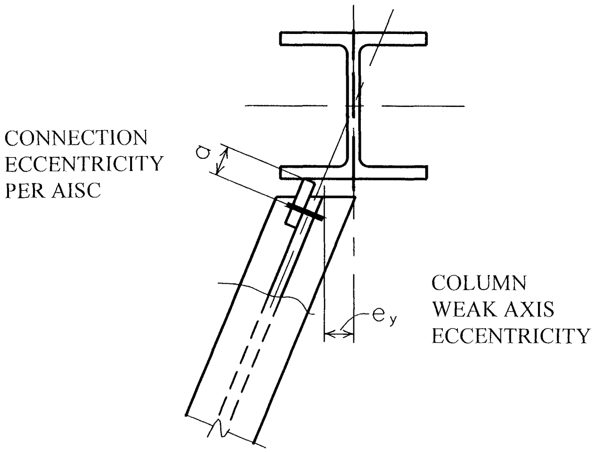


Figure 2.46 Single-plate (shear tab) gravity axis configuration. (*Courtesy of Kloiber and Thornton, with permission from ASCE.*)

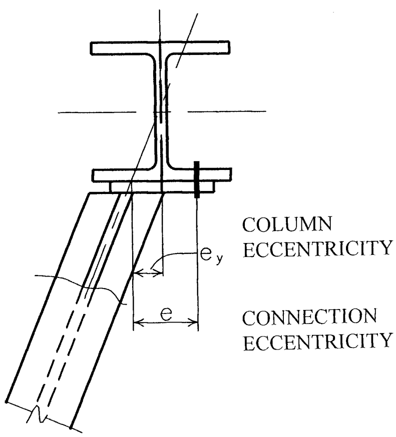


Figure 2.47 Eccentric shear end plate gravity axis configuration. (*Courtesy of Kloiber and Thornton, with permission from ASCE.*)

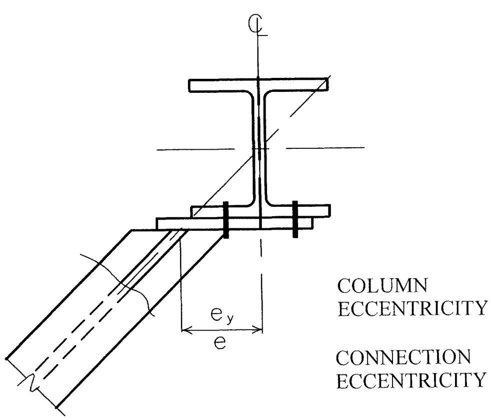


Figure 2.48 Eccentric shear end plate for high skew. (*Courtesy of Kloiber and Thornton, with permission from ASCE.*)

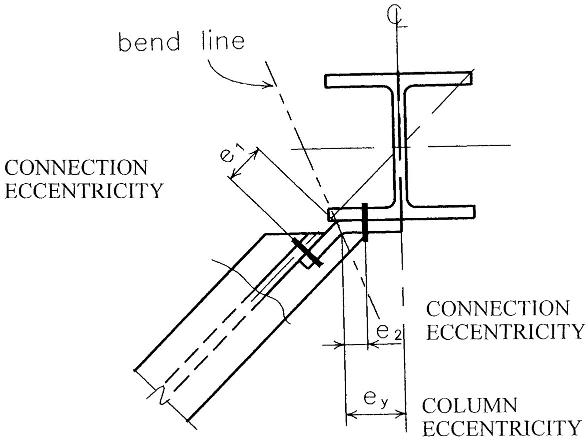


Figure 2.49 Single bent plate—one beam framing to flange. (*Courtesy of Kloiber and Thornton, with permission from ASCE.*)

column weak axis, is present in these connections as shown in Figs. 2.46 through 2.49. The column may need to be designed for this moment.

A special skewed connection is often required when there is another beam framing to the column flange at 90°. If the column flange is not wide enough to accommodate a side-by-side connection, a bent plate can be shop welded to the column with matching holes for a second beam as shown in Fig. 2.50. The plate weld is sized for the eccentricity, e_2, plus any requirement for development of fill plate in the orthogonal connection, and the column sees an eccentric moment due to e_y, which is equal to e_2 in this case.

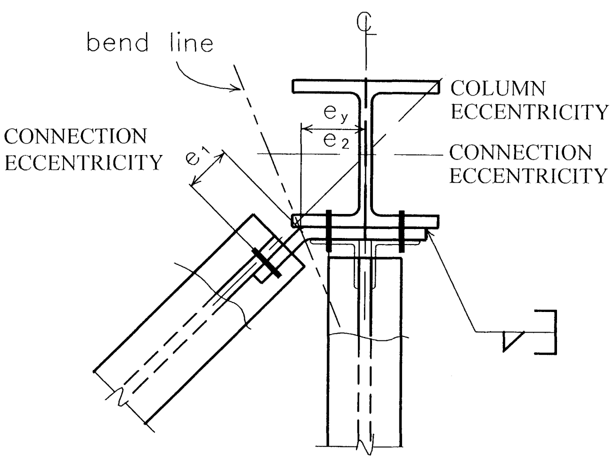


Figure 2.50 Single bent plate—two beams. (*Courtesy of Kloiber and Thornton, with permission from ASCE.*)

2.4.4 Seated connections

The second type of shear connection is the seated connection, either unstiffened or stiffened (Fig. 2.51). As with the framed connections, there are tables in the *Manual of Steel Construction*, which aid in the design of these connections.

The primary use for this connection is for beams framing to column webs. In this case, the seat is inside the flange or nearly so, and is not an architectural problem. It also avoids the erection safety problems associated with most framed connections where the same bolts support beams on both sides of the column web.

When a seat is attached to one side of the column web, the column web is subjected to a local bending pattern because the load from the beam is applied to the seat at some distance, e, from the face of the web. For stiffened seats, this problem was addresses by Sputo and Ellifrit (1991). The stiffened seat design tables (Tables 10-7 and 10-8) in the AISC 13th Edition *Manual of Steel Construction* reflect the results of their

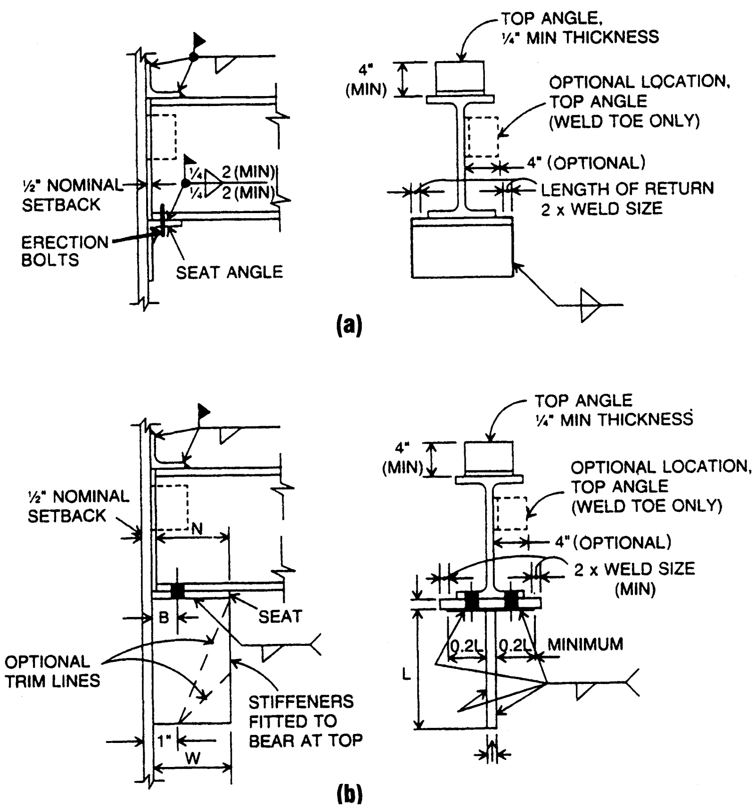


Figure 2.51 Standardized weld seat connections: (*a*) unstiffened seat and (*b*) stiffened seat.

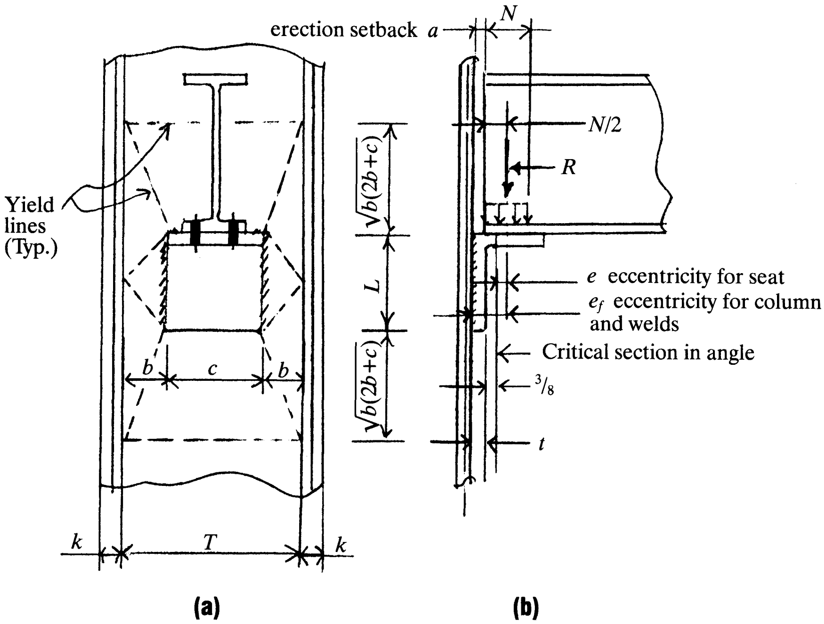


Figure 2.52 Column web yield lines and design parameters for unstiffened seated connection.

research. For unstiffened seats, column web bending also occurs, but no research has been done to determine its effect. This is the case because the loads and eccentricities for unstiffened seats are much smaller than for stiffened seats. Figure 2.52 presents a yield-line analysis that can be used to assess the strength of the column web. The nominal capacity of the column web is

$$R_w = \frac{2m_pL}{e_f}\left(2\sqrt{\frac{T}{b}} + \frac{T}{L} + \frac{L}{2b}\right)$$

where the terms are defined in Fig. 2.52, and

$$m_p = \frac{t_w^2F_y}{4}$$

$$b = \frac{T-c}{2}$$

Since this is a yield limit state, $\phi = 0.9$ and $\Omega = 1.67$.

Example A W14 × 22 beam (A992) is to be supported on an unstiffened seat to a W14 × 90 column (A992). The given reaction (required strength) is 33 kips. Design the unstiffened seat.

The nominal erection set back $a = 1/2$ in. For calculations, to account for underrun, use $a = 3/4$ in. Try a seat 6 in long ($c = 6$). In order to use Table 10-6 from the AISC *Manual of Steel Construction*, the required bearing length, N, must first be determined. Note that N is not the horizontal angle leg length less a, but rather it cannot exceed this value. The bearing length for an unstiffened seat starts at the end on the beam and spreads from this point, because the toe of the angle leg tends to deflect away from the bottom flange of the beam. The bearing length cannot be less than k and can be written in a general way as

$$N = \max\left\{\frac{R - \phi R_1}{\phi R_2}, \frac{R - \phi R_3}{\phi R_4}, \frac{R - \phi R_5}{\phi R_6}, k\right\}$$

where R_1 through R_6 are defined in the AISC *Manual of Steel Construction* pp. 9–48, and are tabulated in Table 9-4. For the W14 × 22,

$$\phi R_1 = 21.1, \phi R_2 = 11.5, \phi R_3 = 23.1, \phi R_4 = 2.86, \phi R_5 = 20.4, \phi R_6 = 3.82$$

Thus

$$N = \max\left\{\frac{33 - 21.1}{11.5}, \frac{33 - 23.1}{2.86} \text{ or } \frac{33 - 20.4}{3.82}, 0.735\right\}$$
$$= \max\{1.04, 3.46 \text{ or } 3.30, 0.735\} = 3.46 \text{ or } 3.30$$

Therefore, N is either 3.46 or 3.30 depending on whether $N/d \leq 0.2$ or $N/d > 0.2$, respectively. With $d = 13.7$, 3.46/13.7 = 0.253, and 3.30/13.7 = 0.241. Since clearly $N/d > 0.2$, N = 3.30 in.

It was stated earlier that $(N + a)$ cannot exceed the horizontal angle leg. Using $a = 1/2 + 1/4 = 3/4$, $N + a = 3.30 + 0.75 = 4.05$, which establishes a required horizontal leg equal to at least 4 in.

The AISC *Manual of Steel Construction* Table 10-6 does not include required bearing lengths greater than $3\frac{1}{4}$ in. However, extrapolating beyond the table, it would seem that a 1-in angle would be an appropriate choice. Since there is no L6 × 4 × 1 available, use a 6 × 6 × 1. The extra length of the horizontal leg is irrelevant. Table 10-6 indicates that a $5/16$ fillet weld of the seat vertical leg (the 6-in leg) to the column web is satisfactory (40.9 kips). Consider this to be a preliminary design, which needs to be checked.

The design strength of the seat angle critical section is

$$\phi R_b = \phi F_y \frac{ct^2}{4e}$$

where the terms are defined in Fig. 2.52. From Fig. 2.52, $e_f = N/2 + a = 3.30/2 + 0.75 = 2.41$ and $e = e_f - t - 0.375 = 2.41 - 1 - 0.375 = 1.04$, $e = 6$. Then

$$\phi R_b = 0.9(36)\frac{(6)(1.0)^2}{4(1.04)} = 46.7 \text{ kips} > 33 \text{ kips, ok}$$

The weld sizes given in Table 10-6 will always be conservative because they are based on using the full horizontal angle leg minus a as the bearing length, N. The detailed check will be performed here for completeness. Using the eccentric weld Table 8-4 with $ex = ef = 2.41$, $l = 6$, $a = {}^{2.41}/_{6} = 0.40$, C is determined to be 2.81. The strength of the weld is calculated as

$$\phi R_{weld} = 0.75(2.81)(5)(6) = 63.2 \text{ kips} > 33 \text{ kips, ok}$$

Finally, checking the column web,

$$\begin{aligned}
m_p &= \frac{50(0.44)^2}{4} = 2.42\frac{\text{kip-in}}{\text{in}} \\
T &= 11.25 \\
c &= 6 \\
L &= 6 \\
b &= \frac{11.25 - 6}{2} = 2.625 \\
\phi R_w &= 0.9\,\frac{2(2.42)(6)}{(2.41)}\left(2\sqrt{\frac{11.25}{2.625}} + \frac{11.25}{6} + \frac{6}{2(2.625)}\right) \\
&= 77.6 \text{ kips} > 33 \text{ kips, ok}
\end{aligned}$$

This completes the calculations for the example. The final design is shown in Fig. 2.53.

2.4.5 Beam shear splices

If a beam splice takes moment as well as shear, it is designed with flange plates in a manner similar to the truss chord splice treated in Sec. 2.2.5.2. The flange force is simply the moment divided by the center-to-center flange distance for inside and outside plate connections, or the moment divided by the beam depth for outside plate connections. The web connection takes any shear. Two typical shear splices are shown in Fig. 2.54. These are common in cantilever roof construction. Figure 2.54a shows a four-clip angle splice. The angles can be shop bolted (as shown) or shop

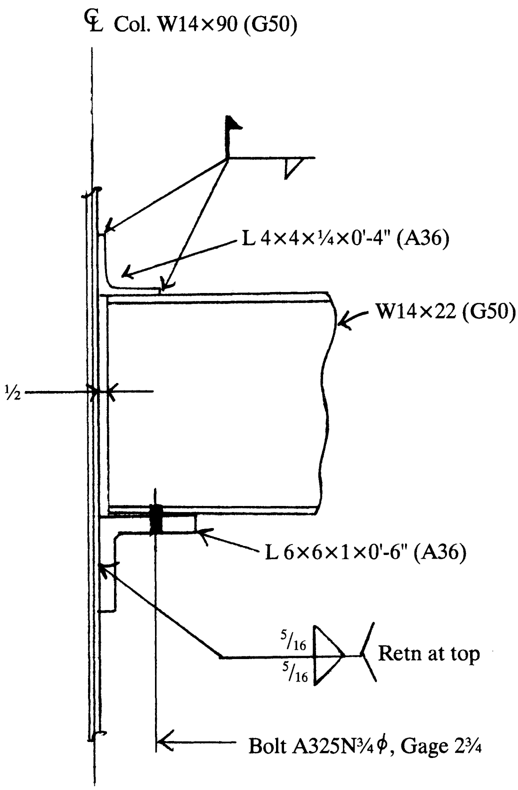


Figure 2.53 Unstiffened seat design.

welded to the beam webs. The design of this splice is exactly the same as that of a double-angle framing connection. The shear acts at the faying surface of the field connection and each side is designed as a double-angle framing connection. If shop bolted all the bolts are in shear only; there is no eccentricity considered on the bolts. If shop welded, the shop welds see an eccentricity from the location of the shear at the field faying surface to the centroids of the weld groups. This anomaly is historical. The bolted connections derive from riveted connections, which were developed before it was considered necessary to satisfy "the niceties of structural mechanics" according to McGuire (1968).

A second type of shear splice uses one or two plates in the plate of the four angles. This type, shown in Fig. 2.54*b*, has moment capacity, but has been used for many years with no reported problems. It is generally less expensive than the angle type. Because it has moment capability, eccentricity on the bolts or welds cannot be neglected. It has been shown by Kulak and Green (1990) that if the stiffness on both sides of the splice is the same, the eccentricity is one-half the distance

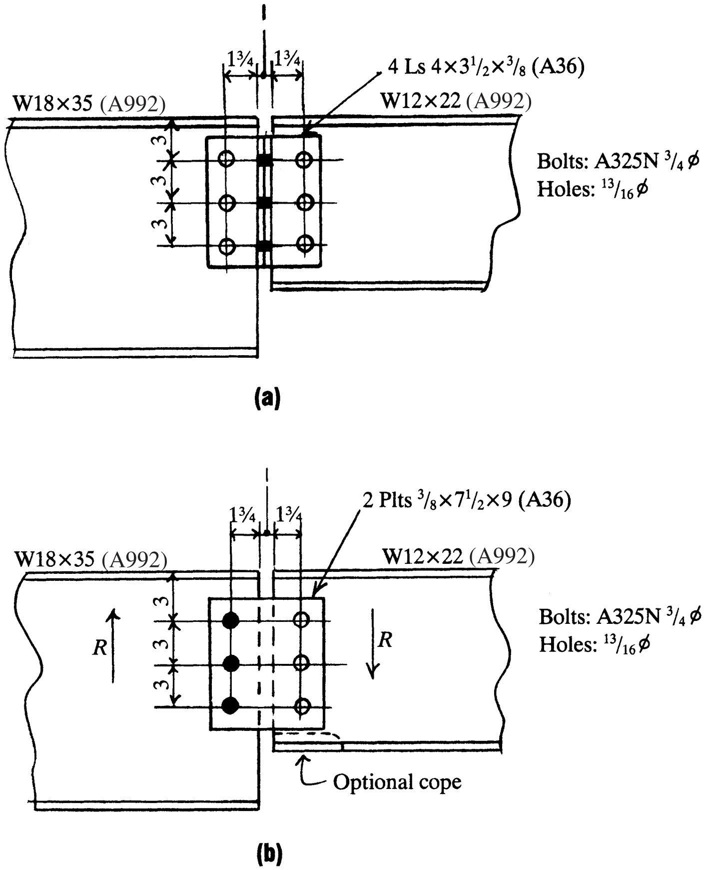


Figure 2.54 Typical shear splices: (*a*) shear splice with four angles and (*b*) shear splice with one or two plates.

between the group centroids, on each side of the splice. This will be the case for a shop-bolted–field-bolted splice as shown in Fig. 2.54*b*. A good discussion on various shear splice configurations and the resulting eccentricities is given in the AISC *Manual of Steel Construction* (2005) pp. 10–129, 131.

Example As an example of the design routine for the Fig. 2.54*b* splice, its capacity (design strength) will be calculated.

Bolts. Since the strength of the bolt group will be determined using Manual Table 7-7 and the direction and magnitude of the force on each bolt will not be known, bolt tearout will be determined based on the worst possible case. This is conservative. A more exact value can be obtained by applying the instantaneous center of rotation method to determine the magnitude and direction of the forces on the individual bolts.

The design "bolt value" will be the minimum of the bolt shear, bearing, and tearout:
Bolt shear

$$\phi r_v = (2)15.9 = 31.8 \text{ kips/bolt}$$

Since the W12 × 22 has the thinner web, it will be checked for bearing and tearout

Bearing at W12 × 22 web

$$\phi r = 0.75(2.4)(65)(0.75)(0.260) = 22.8 \text{ kips/bolt}$$

Tearout at W12 × 22 web (assuming a maximum optional cope depth of $1^1/_2$ in)

$$\phi r = 0.75(1.2)(65)\left(1.75 - \frac{0.813}{2}\right)(0.260) = 20.4 \text{ kips/bolt}$$

Bearing and tearout at splice plates does not govern by inspection.

From Table 7-7, for ex = 2.25 and $n = 3$, $C = 2.11$. Therefore,

$$2.11(20.4) = 43.0 \text{ kips}$$

Neglecting the tearout check as would have been done *prior* to the 3rd of the AISC LRFD *Manual of Steel Construction*, the bolt group capacity would have been 2.11(22.8) = 48.1 kips. The instantaneous center of rotation method assumes that the bolt is the weakest element. However, when the capacity of the group is limited instead by the strength of the connected material, an alternative force distribution can produce an increased calculated capacity (Thornton and Muir, 2004). If the capacity of the bolt group is optimized, the calculated capacity, considering bolt tearout, becomes 46 kips, still a considerable decrease from the capacity neglecting the tearout limit states, but a considerable increase from the 43 kips capacity that results from the worst case.

2.4.6 Extended single plate shear connections (shear tabs)

The 2005 AISC Manual includes specific information relating to the design of extended single-plate connections. For decades the Manual has provided information regarding the design of connections with extended gages, but never included much detail regarding the required checks for such connections. Single-plate shear connections can be very economical connections. In-fill beams can be drilled on the fabricator's drill line with no further handling, since the beams will require none of the coping required for more traditional beam-to-beam connections. Beam-to-column-web connections are also made easier. Since the beam can be connected beyond the column flanges erection is greatly eased. Unlike double angle, end plate and sometimes single angle connections, there will be no common bolts at the support, so safety is also improved.

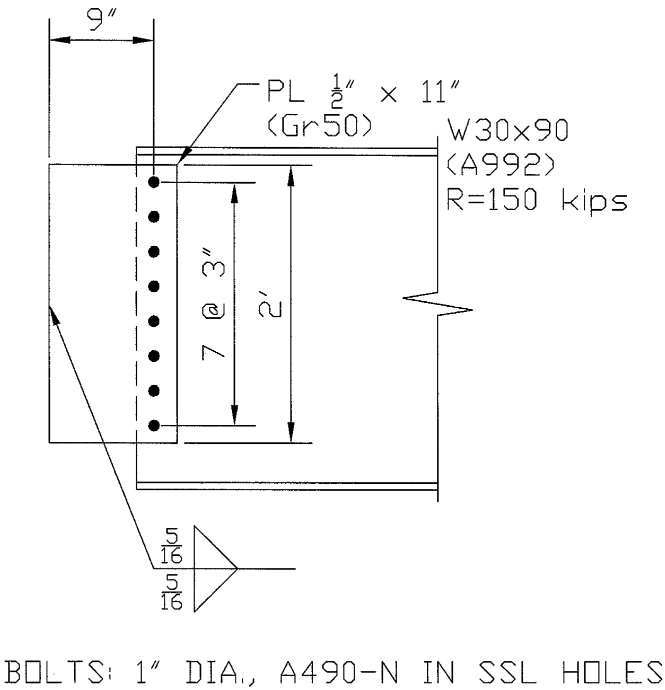


Figure 2.55 Extended Single Plate Connection.

Example—Extended Single Plate Tab Connection (See Fig. 2.55)

Inelastic bolt design. (From AISC 13th Edition Manual Table 7-7)

$$
\begin{aligned}
C &= 4.34 \\
\phi R_n &= C \times \phi r_n \\
&= 4.34 \times 35.3 \\
&= 153 \geq 150 \text{ kips, ok}
\end{aligned}
$$

Bearing / Tearout On Controlling Element
Bearing / Tearout Does Not Control

Maximum Plate Thickness—due to the uncertainty related to the distribution of moments through the connection the plate and bolt group are sized such that yielding in the plate will preclude fracture of the bolts by redistributing the moments. It should be noted that this check uses the nominal bolt capacity without a factor of safety and discounts the 20% reduction in bolt shear strength assumed in the Specification to account for uneven force distribution in end-loaded connections. Since this is essentially a ductility check and not a strength limit state, this should not be considered a violation of the Specification.

Calculating the bolt value as described above:

$$R_n = \frac{\phi_v r_n(1.25)}{\phi_v} = \frac{35.3(1.25)}{0.75} = 58.8 \text{ kips/bolt}$$

$$M = R_n \times C'$$
$$= 58.8 \times 44.5$$
$$= 2620 \text{ kips-in}$$

$$t_{\max} = \frac{6M}{F_y L^2} = \frac{6(2620)}{(50)(24)^2} = 0.546 \geq 0.5 \text{ ok}$$

Gross shear and bending interaction on plate
First the plate is checked to ensure buckling does not control

$$\lambda = \frac{L\sqrt{F_y}}{t_p\sqrt{47{,}500 + 112{,}000\left(\dfrac{L}{2a}\right)^2}}$$

$$= \frac{24\sqrt{50}}{0.5\sqrt{47{,}500 + 112{,}000\left[\dfrac{24}{2(9)}\right]^2}} = 0.683 \leq 0.7$$

Therefore, buckling does not control

$$\phi R_n = \frac{\phi F_y t_p L}{\sqrt{16\left(\dfrac{a}{L}\right)^2 + 2.25}}$$

$$= \frac{0.9(50)(0.5)(24)}{\sqrt{16\left(\dfrac{9}{24}\right)^2 + 2.25}} = 255 \text{ kips} \geq 150 \text{ kips, ok}$$

Net shear on plate

$$\phi R_n = \phi 0.6 F_y t_p\left[L - n\left(\phi_b + \frac{1}{16}\right)\right]$$

$$= (0.75)0.6(65)(0.5)[24 - 8(1.125)] = 219 \text{ kips} \geq 150 \text{ kips, ok}$$

Weld size required – Note the weld size is required to be $^5/_8$ of the plate thickness to ensure that the plate yields and redistributes load prior to weld fracture.

$$w = \frac{5}{8}t_p = \frac{5}{8}(0.5) = 0.3125 \text{ use } ^5/_{16}'' \text{ weld}$$

Block shear on plate

$$A_{nt} = t_p\left[L_e - 0.5\left(\phi_h + \frac{1}{16}\right)\right]$$

$$= 0.5\left[2 - 0.5\left(1.3125 + \frac{1}{16}\right)\right] = 0.656 \text{ in}^2$$

$$A_{gv} = t_p[L_e + (n-1)b] = 0.5[1.5 + (7)3] = 11.3 \text{ in}^2$$

$$A_{nv} = t_p\left[L_e + (n-1)b - (n-0.5)\left(\phi_h + \frac{1}{16}\right)\right]$$

$$= 0.5[1.5 + (7)3 - (7.5)(1.125)] = 7.03 \text{ in}^2$$

$$\begin{aligned} \phi R_{bs} &= \phi(0.6F_uA_{nv} + U_{bs}F_uA_{nt}) \leq \phi(0.6F_yA_{gv} + U_{bs}F_uA_{nt}) \\ &= 0.75[0.6(65)(7.03) + 1.0(65)(0.656)] \\ &= 238 \leq 0.75[0.6(50)(11.3) + 1.0(65)(0.656)] = 284 \\ &= 238 \text{ kips} \geq 150 \text{ kips, ok} \end{aligned}$$

It is generally assumed that beams are torsionally supported at their ends. Lack of torsional support can substantially reduce the flexural capacity of beams that are otherwise laterally unsupported. Generally, the torsional stiffness of end connections to beams that are fully braced by a diaphragm, such as a slab or a deck, is not an issue. However, though the AISC Specification does not contain a check for torsional stiffness, end connections for beams that are not laterally supported should be checked. The check presented here is based on Australian requirements, which assume lateral support only at the applied mid-span load. Assuming the W30 × 90 has a span of 28 ft:

$$k_s \geq 448000\frac{J}{L}\left[1 + \left(\frac{b_f d}{t_f L}\right)^2\right]$$

$$\frac{3730(24)(0.5)^3}{9.0} \geq 448000\frac{2.84}{336}\left[1 + \left(\frac{(10.4)(29.5)}{(0.610)(336)}\right)^2\right]$$

$$1240\ \frac{\text{kips-in}}{\text{radian}} < 12300\ \frac{\text{kips-in}}{\text{radian}}$$

Therefore, the beam cannot be considered to be torsionally restrained by the extended shear tab.

In order to provide sufficient torsional restraint, the shear tab thickness would need to be.

$$t_p = \sqrt[3]{\frac{12300(9.0)}{3730(24)}} = 1.07''$$

Interestingly, even a standard shear tab may not provided adequate torsional restraint in this instance. The required thickness of a standard tab with a 3-in distance from the bolt line to the weld can be calculated as:

$$t_p = \sqrt[3]{\frac{12300(3.0)}{3730(24)}} = 0.744 \text{ in}$$

and at least a $^3/_4''$ shear tab would be required.

2.5 Miscellaneous Connections

2.5.1 Simple beam connections under shear and axial load

As its name implies, a simple shear connection is intended to transfer shear load out of a beam while allowing the beam to act as a simply supported beam. The most common simple shear connection is the double-angle connection with angles shop bolted or welded to the web of the carried beam and field bolted to the carrying beam or column. This section, which is from Thornton (1995a), will deal with this connection.

Under shear load, the double-angle connection is flexible regarding the simple beam end rotation, because of the angle leg thickness and the gage of the field bolts in the angle legs. The AISC 13th Edition Manual, p. 10-9 recommends angle thicknesses not exceeding 5/8 in with the usual gages. Angle leg thicknesses of 1/4 to 1/2 in are generally used, with 1/2-in angles usually being sufficient for the heaviest shear load. When this connection is subjected to axial load in addition to the shear, the important limit states are angle leg bending and prying action. These tend to require that the angle thickness increase or the gage decrease, or both, and these requirements compromise the connection's ability to remain flexible to simple beam end rotation. This lack of connection flexibility causes a tensile load on the upper field bolts, which could lead to bolt fracture and a progressive failure of the connection and the resulting collapse of the beam. It is thought that there has never been a reported failure of this type, but is perceived to be possible.

Even without the axial load, some shear connections are perceived to have this problem under shear alone. These are the single-plate shear connections (shear tabs) and the Tee framing connections. Recent research on the Tee framing connections (Thornton, 1996) has led to a formula (AISC 13th Edition Manual, pp. 9-13, 9-14) which can be used to assess the resistance to fracture (ductility) of double-angle shear connections. The formula is

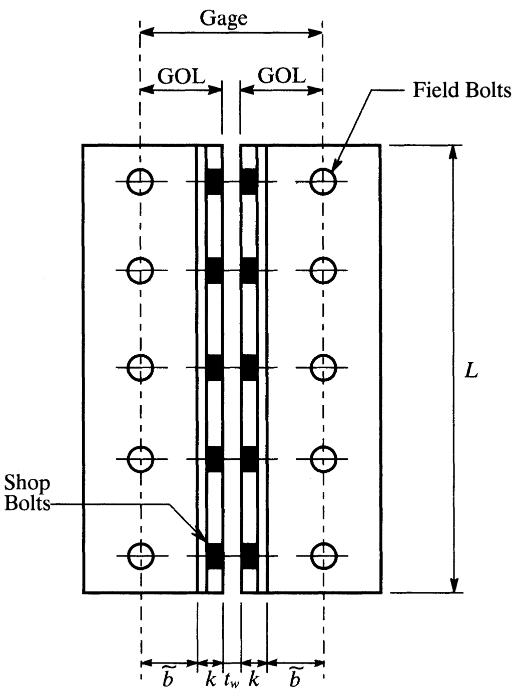
$$d_{b_{\min}} = 0.163t\sqrt{\frac{F_y}{\tilde{b}}\left(\frac{\tilde{b}^2}{L^2} + 2\right)}$$

where $d_{b_{\min}}$ = the minimum bolt diameter (A325 bolts) to preclude bolt fracture under a simple beam end rotation of 0.03 radians

t = the angle leg thickness

$\tilde{b}$ = the distance from the bolt line to the k distance of the angle (Fig. 2.56)
L = the length of the connection angles.

Note that this formula can be used for ASD and LRFD designs in the form given here. It can be used to develop a table (Table 2.1) of angle thicknesses and gages for various bolt diameters which can be used as a guide for the design of double-angle connections subjected to shear and axial tension. Note that Table 2.1 validates AISC's long-standing (AISC,



Fgure 2.56 Geometry of double angles (shop-bolted shown).

TABLE 2.1 Estimated Minimum Angle Gages (GOL) for A36 Angles and A325 Bolts for Rotational Flexibility

Angle thickness (in)	Minimum gage of angle (GOL)[a] $^3/_4$-in-diameter bolt (in)	$^7/_8$-in-diameter bolt (in)	1-in-diameter bolt (in)
$^3/_8$	$1^3/_8$	$1^1/_4$	$1^1/_8$
$^1/_2$	$1^7/_8$	$1^5/_8$	$1^1/_2$
$^5/_8$	$2^1/_2$	$2^1/_8$	$1^7/_8$
$^3/_4$	$3^1/_4$	$2^{11}/_{16}$	$2^5/_{16}$
1	6	$4^5/_{16}$	$3^1/_2$

[a] Driving clearances may control minimum GOL.

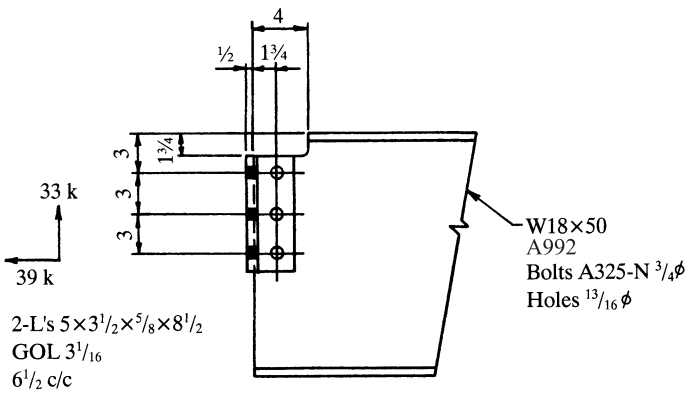


Figure 2.57 Framed connection subjected to axial and shear loads.

1970) recommendation (noted above) of a maximum $^5/_8$-in angle thickness for the "usual" gages. The usual gages would be $4^1/_2$ to $6^1/_2$ in. Thus, for a carried beam web thickness of say $^1/_2$ in, GOL will range from 2 to 3 in. Table 2.1 gives a GOL of $2^1/_2$ in for $^3/_4$-in bolts (the most critical as well as the most common bolt size). Note also that Table 2.1 assumes a significant simple beam end rotation of 0.03 radians, which is approximately the end rotation that occurs when a plastic hinge forms at the center of the beam. For short beams, beams loaded near their ends, beams with bracing gussets at their end connections, and beams with light shear loads, the beam end rotation will be small and Table 2.1 does not apply.

As an example of a double-clip angle connection, consider the connection of Fig. 2.57. This connection is subjected to a shear load of 33 kips and an axial tensile load of 39 kips.

Shop bolts. The shop bolts "see" the resultant load $R = \sqrt{33^2 + 39^2} =$ 51.1 kips. The design shear strength of one bolt is $\phi r_v = 15.9$ kips in single shear, and 31.8 kips in double shear.

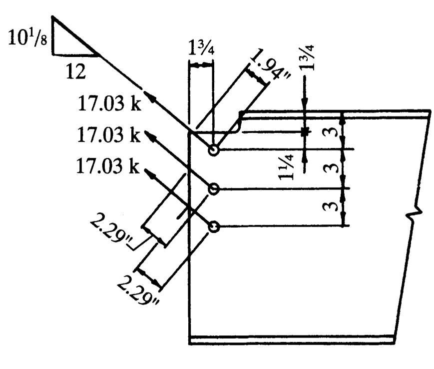


Figure 2.58 Edge distances along the line of action.

The beam web will govern the bearing and tearout values. The bearing strength is

$$\phi r_p = 0.75(2.4)(0.75)(0.355)(65) = 31.2 \text{ kips}$$

If the loads of 33 kips shear and 39 kips axial always remain proportional, that is, maintain the bevel of $10^1/_8$ to 12 as shown in Fig. 2.58, the spacing requirement is irrelevant because there is only one bolt in line of force and the true edge distance is 1.94 or 2.29 in. The clear distance, L_C, is

$$L_c = 1.94 - (0.8125/2) = 1.53 \text{ in}$$

In order to maintain equilibrium all bolts will be assumed to have the same strength based on this shortest edge distance the tearout capacity is

$$\phi r_v = 0.75(1.2)(1.53)(0.355)(65) = 31.8 \text{ kips}$$

The capacity of the bolt group is

$$\phi R_v = 3(31.2) = 93.6 \text{ kips} > 51.1 \text{ kips, ok}$$

Gross shear on clips is

$$\phi R_n = 1.0(0.6)(36)(8.5)(0.625)(2) = 230 \text{ kips} > 33 \text{ kips, ok}$$

Net shear on clips is

$$\begin{aligned}\phi R_n &= 0.75(0.6)(58)[8.5 - 3(0.875)](0.625)(2) \\ &= 192 \text{ kips} > 33 \text{ kips, ok}\end{aligned}$$

Block shear rupture (tearout). A simple conservative way to treat block shear when shear and tension are present is to treat the resultant as a shear. Then, from Figs. 2.57 and 2.58,

$$\begin{aligned}A_{gv} &= 7.25 \times 0.355 = 2.57 \text{ in}^2 \\ A_{nv} &= (7.25 - 2.5 \times 0.875) \times 0.355 = 1.80 \text{ in}^2 \\ A_{gt} &= 1.75 \times 0.355 = 0.621 \text{ in}^2 \\ A_{nt} &= (1.75 - 0.5 \times 0.875) \times 0.355 = 0.466 \text{ in}^2 \\ F_u A_{nt} &= 65 \times 0.466 = 30.3 \\ \phi R_{bsv} &= 0.75\,[F_u A_{ntv} + \min(0.6 F_y A_{gtv}, 0.6 F_u A_{ntv})] \\ &= 0.75\,[30.3 + \min(0.6 \times 50 \times 2.57, 0.6 \times 65 \times 1.80)] \\ &= 75.4 \text{ kips} > 51.1 \text{ kips, ok}\end{aligned}$$

An alternate approach is to calculate a block shear rupture design strength under tensile axial load. From Fig. 2.60,

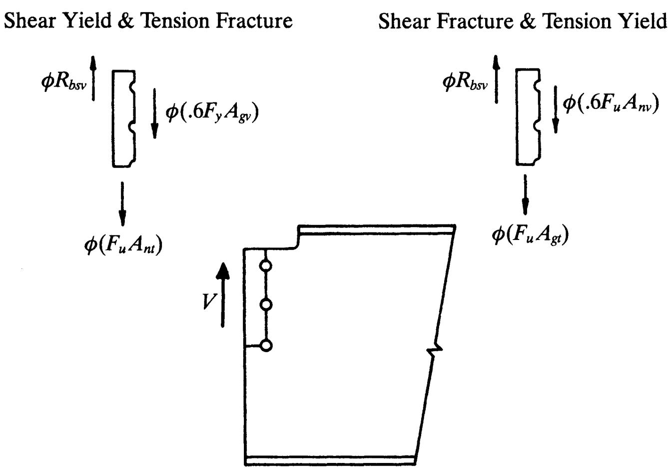


Figure 2.59 Block shear rupture under shear.

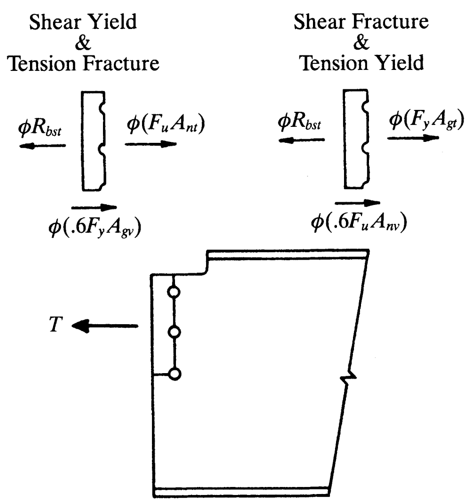


Figure 2.60 Block shear rupture under tension T.

$$
\begin{aligned}
A_{gv} &= 0.621 \text{ in}^2 \\
A_{nv} &= 0.466 \text{ in}^2 \\
A_{gt} &= 2.57 \text{ in}^2 \\
A_{nt} &= 1.80 \text{ in}^2 \\
F_uA_{nt} &= 65 \times 1.80 = 117 \\
\phi R_{bsv} &= 0.75\,[F_uA_{\text{ntv}} + \min(0.6F_yA_{\text{gtv}}\,,\ 0.6F_uA_{\text{ntv}}\,)] \\
&= 0.75\,[\,117 + \min(0.6 \times 50 \times 0.621,\ 0.6 \times 65 \times 0.466)] \\
&= 101 \text{ kips}
\end{aligned}
$$

Using an elliptical interaction equation, which is analogous to the von Mises (distortion energy) yield criterion,

$$\left(\frac{V}{\phi R_{bsv}}\right)^2 + \left(\frac{T}{\phi R_{bst}}\right)^2 \leq 1$$

where V is the factored shear and T the factored tension. Then

$$\left(\frac{33}{75.4}\right)^2 + \left(\frac{39}{101}\right)^2 = 0.34 < 1 \text{ ok}$$

This interaction approach is always less conservative than the approach using the resultant $R = \sqrt{V^2 + T^2}$ as a shear because $\phi R_{bst} > \phi R_{bsv}$ for the geometries of the usual bolt positioning in double-angle connections with two or more bolts in a single vertical column. The resultant approach, being much simpler as well as conservative, is the method most commonly used.

Connection angles. Figure 2.57 shows angles $5 \times 3^1/_2 \times {}^5/_8$, but assume for the moment that 1/4 angles are to be checked. The shop legs are checked for the limit states of bearing, gross shear and gross tension, and net shear and net tension. Net shear rupture and net tension rupture will control over block shear rupture with the usual connection geometries, that is, $1^1/_4$ edge and $1^1/_4$ end distances. Since the sum of the clip angle thicknesses $= 0.24 + 0.25 = 0.5 > 0.355$, the beam web and not the shop legs of the clip angles will control.

Prying action. The AISC LRFD Manual has a table to aid in the selection of a clip angle thickness.

The preliminary selection table, Table 15.1, indicates that a 5/8 angle will be necessary. Trying Ls $5 \times 3^1/_2 \times 5/8$, and following the procedure of the AISC Manual,

$$b = \frac{6.5 - 0.355 - 2 \times 0.625}{2} = 2.45$$

$$a = \frac{10.355 - 6.5}{2} = 1.93 (< 1.25 \times 2.45 = 3.06 \text{ ok})$$

$$b' = 2.45 - \frac{0.75}{2} = 2.08$$

$$a' = 1.93 + \frac{0.75}{2} = 2.31$$

$$\rho = \frac{2.08}{2.31} = 0.90$$

$$p = \frac{8.5}{3} = 2.83$$

$$\delta = 1 - 0.8125/2.83 = 0.71$$

The shear per bolt $V = 33/6 = 5.5$ kips <15.9 kips, ok. The tension per bolt $T = 39/6 = 6.5$ kips. Because of interaction,

$$\phi F_t' \; 5 \; 0.75\left[1.3F_{nt} - \left(\frac{F_{nt}}{\phi F_{nv}}\right)f_v\right] \leq \phi F_{nt}$$

With $f_v = 5.5/0.4418 = 12.5$ ksi,

$$\phi F_t' = 0.75\left[1.3 \times 90 - \left(\frac{90}{0.75 \times 48}\right)12.5\right]64.3 \text{ ksi} > 67.5 \text{ ksi}$$

Use $\phi F_t' = 64.3$ ksi, and $\phi r_t' = 64.3 \times 0.4418 = 28.4$ kips/bolt. Since $T = 6.5$ kips < 28.4 kips, the bolts are satisfactory independent of prying action. Returning to the prying action calculation

$$t_c = \sqrt{\frac{4.44 \times 28.4 \times 2.08}{2.83 \times 58}} = 1.60 \text{ in}$$

$$\alpha' = \frac{1}{0.71 \times 1.90}\left[\left(\frac{1.60}{0.625}\right)^2 - 1\right] = 4.11$$

Since $\alpha' = 4.11$, use $\alpha' = 1$. This means that the strength of the clip angle legs in bending is the controlling limit state. The design strength is

$$T_d = 28.4\left(\frac{0.625}{1.60}\right)^2(1 + 0.71) = 7.41 \text{ kips} > 6.5 \text{ kips, ok}$$

The Ls $5 \times 3^1/_2 \times {}^5/_8$ are satisfactory.

Ductility considerations. The $^5/_8$-in angles are the maximum thickness recommended by the AISC Manual, p. 10-9, for flexible shear connections. Using the formula introduced at the beginning of this section,

$$d_{b_{\min}} = 0.163t\sqrt{\frac{F_y}{\tilde{b}}\left(\frac{\tilde{b}^2}{L^2} + 2\right)}$$

with $t = 0.625$, $F_y = 36$, $\tilde{b} = 3.0625 - 1.125 = 1.94$, $L = 8.5$

$$d_{b_{\min}} = 0.163 \times .625\sqrt{\frac{36}{1.94}\left(\frac{1.94^2}{8.5^2} + 2\right)} = 0.63 \text{ in}$$

Since the actual bolt diameter is 0.75 in, the connection is satisfactory for ductility.

As noted before, it may not be necessary to make this check for ductility. If the beam is short, is loaded near its ends, or for other reasons is not likely to experience very much simple beam end rotation, this ductility check can be omitted.

This completes the calculations for the design shown in Fig. 2.57.

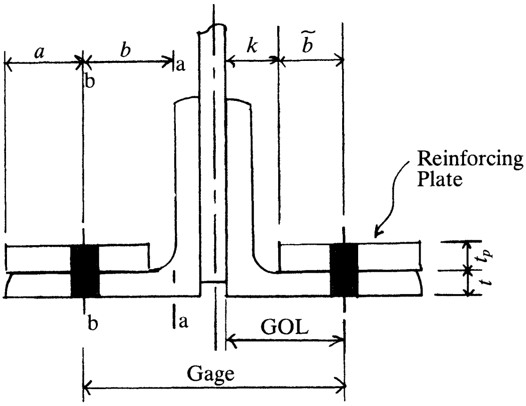


Figure 2.61 Prying action with reinforcing (washer) plate.

2.5.2 Reinforcement of axial force connections

It sometimes happens that a simple beam connection, designed for shear only, must after fabrication and erection be strengthened to carry some axial force as well as the shear. In this case, washer plates can sometimes be used to provide a sufficient increase in the axial capacity. Figure 2.61 shows a double-angle connection with washer plates that extend from the toe of the angle to the *k* distance of the angle. These can be made for each bolt, so only one bolt at a time need be removed, or if the existing load is small, they can be made to encompass two or more bolts on each side of the connection. With the washer plate, the bending strength at the "stem" line, section a-a of Fig. 2.61 is

$$M_n = \frac{1}{4} F_u pt^2$$

while that at the bolt line, section b-b, is

$$M_n{}' = \alpha\delta \frac{1}{4} F_u p\,(t^2 + t_\mathrm{p}^2) = \alpha\delta \frac{1}{4} F_u pt^2 \left(1 + \frac{t_p^2}{t^2}\right) = \alpha\delta\eta M_n$$

where $\eta = 1 + \left(\frac{t_p}{t}\right)^2$ and the remaining quantities are in the notation of the AISC 13th Edition Manual (2005). With the introduction of η, the prying action formulation of the AISC Manual can be generalized for washer plates by replacing δ wherever it appears by the term dη. Thus

$$\alpha' = \frac{1}{\delta\eta(1 + \rho)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right]$$

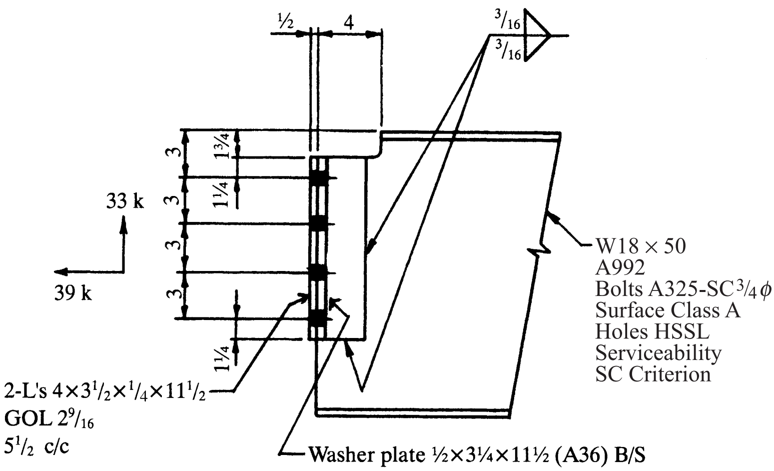


Figure 2.62 A shear connection needing reinforcement to carry axial load of 39 kips.

and

$$T_d = \phi r_t \left(\frac{t}{t_c}\right)^2 (1 + \alpha'\delta\eta)$$

All other equations remain the same.

As an example of the application of this method, consider the connection of Fig. 2.62. Assume this was designed originally for a shear of 60 kips, but now must carry an axial force of 39 kips when the shear is at 33 kips. Let us check the axial capacity of this connection. The most critical limit state is prying action because of the thin angle leg thickness. From Fig. 2.62

$$b = \frac{5.5 - 0.355 - 0.25}{2} = 2.45$$

$$a = \frac{8 + 0.355 - 5.5}{2} = 1.43$$

$$1.25 \times 2.45 = 3.06 > 1.43$$

Use $a = 1.43$. Then $b' = 2.08$, $a' = 1.81$, $\rho = 1.15$, $\delta = 0.72$, $V = 33/8 = 4.125$ kips/bolt. The holes are HSSL (horizontal short slots), so $\phi r_v = 9.41$ kips/bolt. Since $4.125 < 9.41$, the bolts are ok for shear (as they obviously must be since the connection was originally designed for 60 kips shear). Because this is a shear connection, the shear capacity is reduced by the tension load by the factor $1\text{-}T/(1.13T_b)$, where T is the applied load per bolt and T_b is the specified pretension.

Thus, the reduced shear design strength is

$$\phi r_v' = \phi r_v \left(1 - \frac{T}{1.13T_b}\right)$$

This expression can be inverted to a form usable in the prying action equations as

$$\phi r_t' = 1.13T_b\left(1 - \frac{V}{\phi r_v}\right) \le \phi r_t$$

For the present problem

$$\phi r_t' = 1.13 \times 28\left(1 - \frac{4.125}{9.41}\right) = 17.8 \text{ kips} < 29.8 \text{ kips}$$

Use $\phi r_t' = 17.8$ kips. Since $T = 39/8 = 4.875$ kips < 17.8 kips, the bolts are ok for tension/shear interaction exclusive of prying action. Now, checking prying action, which includes the bending of the angle legs,

$$t_C = \sqrt{\frac{4.44 \times 17.8 \times 2.08}{2.875 \times 58}} = 0.993$$

$$\alpha' = \frac{1}{0.72 \times 2.15}\left[\left(\frac{0.993}{0.25}\right)^2 - 1\right] = 9.55$$

Since $\alpha' > 1$, use $\alpha' = 1$, and

$$T_d = 17.8\left(\frac{0.25}{0.993}\right)^2(1.72) = 1.94 \text{ kips} < 4.875 \text{ kips, no good}$$

Thus, the 1/4-in angle legs fail. Try a 1/2-in washer plate. Then

$$\eta = 1 + \left(\frac{0.5}{0.25}\right)^2 = 5.00$$

$$\alpha' = \frac{1}{0.72 \times 5.00 \times 2.15}\left[\left(\frac{0.993}{0.25}\right)^2 - 1\right] = 1.91$$

Since $\alpha' > 1$ use $\alpha' = 1$

$$T_d = 17.8\left(\frac{0.25}{0.993}\right)^2(1 + 0.72 \times 5.00) = 5.19 \text{ kips} > 4.875 \text{ kips, ok}$$

Therefore, the 1/2-in washer plates enable the connection to carry $5.19 \times 8 = 41.5$ kips > 39 kips, ok.

If ductility is a consideration, the ductility formula can be generalized to

$$d_{b_{\min}} = 0.163t\sqrt{\eta}\sqrt{\frac{F_y}{\tilde{b}}\left(\frac{\tilde{b}^2}{L^2} + 2\right)}$$

With $\tilde{b} = GOL - k = 2\frac{9}{16} - \frac{13}{16} = 1.75$

$$d_{b_{\min}} = 0.163 \times 0.25\sqrt{5.00}\sqrt{\frac{36}{1.75}\left(\frac{1.75^2}{11.5^2} + 2\right)} = 0.588 \text{ in} < 0.75 \text{ in, ok}$$

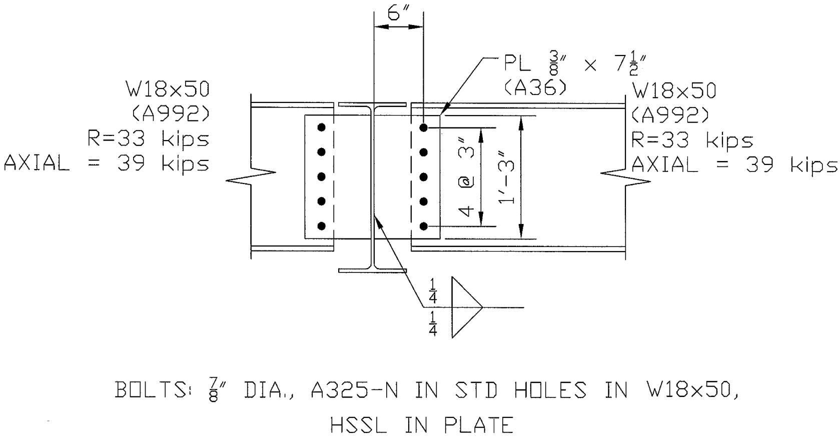


Figure 2.63 Extended Single Plate Connection With Axial Load.

2.5.3 Extended tab with axial

An alternative to the connection shown in Sec. 2.5.1 would be to use an extended tab designed to carry the axial force as shown in Fig. 2.63.

$$\text{Resultant load} = (V^2 + T^2)^{0.5} = 51.1 \text{ kips}$$

$$\phi = \tan^{-1}(39/33) = 49.8°$$

Inelastic bolt design. From AISC Manual Table 7-7at 45° >, ex = 6, n = 5

$$C = 2.88$$

$$\phi R_n = C \times \phi r_n$$

$$= 2.88 \times 21.6$$

$$= 62.2 \geq 51.1 \text{ kips, ok}$$

Bearing / Tearout on controlling element. Bearing / tearout does not control

Maximum plate thickness. The transfer of the axial force is clear. However there are still uncertainties about the distribution of eccentricities so it is recommended to maintain the ductility requirement relating bolt strength to plate strength.

The ultimate bolt capacity can be calculated as

$$R_n = \frac{\phi_v r_n (1.25)}{\phi_v} = \frac{21.6(1.25)}{0.75} = 36 \text{ kips/bolt}$$

$$M = R_n C' \text{ (From AISC Manual Table 7-7 at 0°) >, } C' = 17.1$$

$$= 36.0 \times 17$$

$$= 616 \text{ kips/in}$$

$$t_{\max} = \frac{6M}{F_y L^2} = \frac{6(616)}{(36)(15)^2} = 0.456 \geq 0.375, \qquad \text{ok}$$

Gross shear, axial, and bending interaction on plate. Axial capacity (compresssion). It is conservatively assumed that the beam is not restrained from moving laterally. In many instances, the presence of a composite slab will provide restraint. In such cases, the use of $K = 0.65$ will be more appropriate.

$$K = 1.2$$

$$r = \frac{t_p}{\sqrt{12}} = \frac{0.375}{\sqrt{12}} = 0.108$$

$$\frac{Ka}{r} = \frac{1.2(6)}{0.108} = 66.7$$

$$\phi F_{cr} = 25.7$$

From AISC Manual Table 4-22.

$$\phi R_c = \phi F_{cr} L t_p = 25.7(15)(0.375) = 145 \text{ kips} > 39 \text{ kips, ok}$$

Bending capacity.

$$\lambda = \frac{L\sqrt{F_y}}{t_p\sqrt{47{,}500 + 112{,}000\left(\dfrac{L}{2a}\right)^2}}$$

$$= \frac{15\sqrt{36}}{0.375\sqrt{47{,}500 + 112{,}000\left[\dfrac{15}{2(6)}\right]^2}} = 0.509 \leq 0.7$$

Flexual buckling does not control

$$F_{cr} = F_y Q = 36$$

$$\phi M_b = \frac{\phi F_y t_p L^2}{4} = \frac{0.9(36)(0.375)(15)^2}{4} = 683 \text{ kips-in} > 33 \times 6$$

$$= 198 \text{ kips-in} \qquad \text{ok}$$

Shear capacity.

$$\phi R_v = \phi 0.6 F_y t_p L = 1.0(0.6)(36)(0.375)(15) = 122 \text{ kips-in}$$

Interaction Generally AISC does not require interaction between shear and normal stresses to be checked. However, including this interaction more accurately predicts test results, so it is included here. Additionally, the interaction equations from Chapter H of the Specification are used to combine the effects of the axial and bending forces.

$$\text{Since } \frac{P}{\phi R_c} = \frac{39}{145} = 0.269 \geq 0.2 \text{ Use (H1-1a AISC Manual)}$$

$$\left(\frac{P}{\phi R_c} + \frac{8}{9}\frac{Va}{\phi M_b}\right)^2 + \left(\frac{V}{\phi R_v}\right)^2$$
$$= \left(\frac{39}{145} + \frac{8}{9}\frac{33(6)}{683}\right)^2 + \left(\frac{33}{122}\right)^2 = 0.329 \leq 1.0 \text{ ok}$$

Net shear, axial, and bending interaction on plate. Axial capacity (tension).

$$\phi R_t = \phi F_u t_p [L - n(\phi_b + 1/16)]$$
$$= 0.75(58)(0.375)[15 - 4(0.875)] = 163 \text{ kips}$$

Bending capacity. The net bending capacity is assumed to be the same as the gross bending capacity. This is based on testing reported by Mohr (2005).

$$\phi M_b = 683 \text{ kips-in}$$

Shear capacity.

$$\phi R_t = \phi 0.6 F_u t_p [L - n(\phi_b + 1/16)]$$
$$= 0.75(0.6)(58)(0.375)[15 - 4(0.875)] = 97.9 \text{ kips}$$

Interaction.

$$\text{Since } \frac{P}{\phi R_t} = \frac{39}{163} = 0.239 \geq 0.2 \text{ use (H1-1a AISC Manual)}$$

$$\left(\frac{P}{\phi R_c} + \frac{8}{9}\frac{Va}{\phi M_b}\right)^2 + \left(\frac{V}{\phi R_v}\right)^2$$
$$= \left(\frac{39}{163} + \frac{8}{9}\frac{33(6)}{683}\right)^2 + \left(\frac{33}{97.9}\right)^2 = 0.361 \leq 1.0 \text{ ok}$$

Weld capacity.

$$\theta = \tan^{-1}\left(\frac{T}{V}\right) = \tan^{-1}\left(\frac{39}{33}\right) = 49.8°$$
$$\phi R_w = 2(1.392)LD(1 + 0.5\sin^{1.5}\theta)$$
$$= 2(1.392)(15)(4)[1 + 0.5\sin^{1.5}(49.8)] = 223 \text{ kips} \geq 51.1 \text{ kips, ok}$$

Block shear on plate. L-shaped tearout.

Check block shear due to shear load.

$$A_{nt} = t_p[L_e - 0.5(\phi_h + 1/16)] = 0.375[1.5 - 0.5(1.125 + 1/16)]$$
$$= 0.340 \text{ in}^2$$
$$A_{gv} = t_p[L_e + (n-1)b] = 0.375[1.5 + (5-1)3] = 5.06 \text{ in}^2$$
$$A_{nv} = t_p[L_e + (n-1)b - (n-0.5)(\phi_h + 0.0625)]$$
$$= 0.375[1.5 + (5-1)3 - (5-0.5)(1)] = 3.38 \text{ in}^2$$
$$\phi R_{bsv} = \phi(0.6F_uA_{nv} + U_{bs}F_uA_{nt}) \le \phi(0.6F_yA_{gv} + U_{bs}F_uA_{nt})$$
$$= 0.75[0.6(58)(3.38) + 1.0(58)(0.340)]$$
$$= 103 \le 0.75[0.6(36)(5.06) + 1.0(58)(0.340)] = 96.8$$
$$= 96.8 \text{ kips} > 733 \text{ kips} \quad \text{ok}$$

Check block shear due to axial load.

$$A_{nt} = t_p[L_e + (n-1)b - (n-0.5)(\phi_h + 1/16)]$$
$$= 0.375[1.5 + (4)3 - (4.5)(1.0)]$$
$$= 3.38 \text{ in}^2$$
$$A_{gv} = t_P L_e$$
$$A_{nt} = 0.375 \times 1.5 = 0.563$$
$$A_{nt} = t_p[L_e - 0.5(\phi_h + 1/16)]$$
$$= 0.375[1.5 - 0.5(1.1875)] = 0.339 \text{ in}^2$$
$$\phi R_{bst} = \phi(0.6F_uA_{nv} + U_{bs}F_uA_{nt}) \le \phi(0.6F_yA_{gv} + U_{bs}F_uA_{nt})$$
$$= 0.75[0.6(58)(0.339) + 1.0(58)(3.38)]$$
$$= 156 \le 0.75[0.6(36)(0.563) + 1.0(58)(3.38)] = 156$$
$$= 156 \text{ kips} > 39 \text{ kips} \quad \text{ok}$$

Check combined shear and axial block shear.

$$\left(\frac{V}{\phi R_{bsv}}\right)^2 + \left(\frac{T}{\phi R_{bst}}\right)^2 = \left(\frac{33}{96.8}\right)^2 + \left(\frac{39}{156}\right)^2 = 0.179 \le 1.0 \text{ ok}$$

U - shaped tearout.

$$A_{nt} = t_p[(n-1)(b - (\phi_h + 1/16))]$$
$$= 0.375[(5-1)(3-(1.0))] = 3.00 \text{ in}^2$$
$$A_{gv} = 2t_pL_e = 2(0.375)(1.5) = 1.13 \text{ in}^2$$
$$A_{nv} = 2t_p[L_e - 0.5(\phi_h + 1/16)]$$
$$= 2(0.375)[1.5 - 0.5(1.19)] = 0.679 \text{ in}^2$$

$$\begin{aligned}\phi R_{bst} &= \phi(0.6F_uA_{nv} + U_{bs}F_uA_{nt}) \leq \phi(0.6F_yA_{gv} + U_{bs}F_uA_{nt})\\ &= 0.75[0.6(58)(0.679) + 1.0(58)(3.00)]\\ &= 148 \leq 0.75[0.6(36)(1.13) + 1.0(58)(3.00)] = 149\\ &= 149 \text{ kips} \geq 39 \text{ kips, ok}\end{aligned}$$

Block shear on beam web due to axial load.

$$\begin{aligned}A_{nt} &= t_p[(n - 1)(b - (\phi_h + 1/16))]\\ &= 0.355[(5 - 1)(3 - (1.0))] = 2.84 \text{ in}^2\\ A_{gv} &= 2t_pL_e = 2(0.355)(1.5) = 1.07 \text{ in}^2\\ A_{nv} &= 2t_p[L_e - 0.5(\phi_h + 1/16)]\\ &= 2(0.355)[1.5 - 0.5(1.1875)] = 0.643 \text{ in}^2\\ \phi R_{bst} &= \phi(0.6F_uA_{nv} + U_{bs}F_uA_{nt}) \leq \phi(0.6F_yA_{gv} + U_{bs}F_uA_{nt})\\ &= 0.75[0.6(65)(0.643) + 1.0(65)(2.84)]\\ &= 157 \leq 0.75[0.6(50)(1.07) + 1.0(65)(2.84)] = 163\\ &= 157 \text{ kips} \geq 39 \text{ kips, ok}\end{aligned}$$

There is no block shear limit state on the beam web. This completes the calculations for this example.

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Chapter

3

Welded Joint Design and Production

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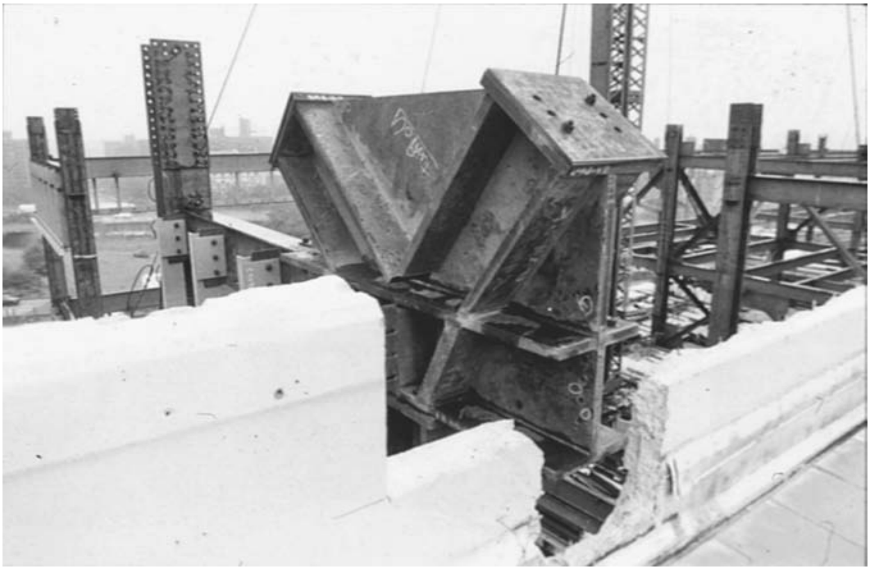


(*Courtesy of The Steel Institute of New York.*)

3.1 Structural Steels for Welded Construction

3.1.1 Introduction

When selecting steels for structural applications, engineers usually select the specific steel based upon the mechanical property of strength, whether yield or tensile. These mechanical properties, along with the modulus of elasticity, in general satisfy the requirements for structural considerations. Additional requirements, such as notch toughness (typically measured by the Charpy V-notch specimen), may be specified to minimize brittle fracture, especially for dynamically loaded structures. With the single exception of weathering steel for uncoated applications exposed to atmospheric conditions, the chemical composition generally is of little concern from a structural point of view.

In terms of weldability, the chemistry of the steel is at least as important as, and arguably more important than, the mechanical properties. During the thermal cutting processes associated with construction, as well as when it is arc-welded, steel is subject to a variety of thermal cycles which can alter its mechanical properties in the area immediately adjacent to the weld metal. This area is known as the *heat-affected zone* and is, by definition, base metal that has been thermally affected by the welding or cutting process. While this region is generally small (typically 2 to 3 mm wide), it may exhibit different strength properties, and the toughness in this region may be dramatically altered. Finally, the base metal composition may have a significant effect on the weld metal composition, particularly for single-pass welds and when welding procedures are used that result in deep penetration. The chemistry of the base material is of the utmost importance in determining the suitability of a steel for welded construction. The mechanical properties of steel cannot be overlooked as they relate to welding, however. The strength of the steel will, in many cases, determine the required strength level of the weld deposit.

Although most steels can be welded, they are not all welded with the same degree of ease. *Weldability* is the term used to describe how readily the steel can be welded. For new construction, it is always advisable to select steels with good weldability since this will inevitably lead to both high-quality and economical construction. Steels with reduced weldability may require specialized electrodes, techniques, preheat and postheat treatments, and joint designs. While materials that are difficult to weld are successfully fabricated every day, the use of these materials is generally inappropriate for new construction given the variety of readily available steels with excellent weldability. Occasionally, the engineer is faced with the situation of welding on material with less than optimum weldability, such as when welding on existing structures is necessary. Under these circumstances, it is advisable to proceed with caution, reviewing the principles outlined in this chapter, and, when necessary, to contact a welding engineer with expertise in metallurgy to address the specifics of the situation.

For most applications, however, modern welding codes such as the American Welding Society's *AWS D1.1 Structural Welding Code—Steel* and the American Institute of Steel Construction's *Steel Construction Manual* list weldable steels suitable for construction. These materials have a long history of satisfactory performance, and the codes supply appropriate guidelines as to what precautions or techniques are appropriate for certain materials. For example, AWS D1.1 lists "prequalified steels" that may be used in conjunction with a prequalified welding procedure. The code requirements for the fabrication of these steels are sufficiently justified that the contractor is not required to qualify the welding procedures by test when using this particular material, provided that all the other prequalified requirements were met.

Codes, however, do not necessarily include new developments from the steel producers. An inevitable characteristic of codes is that they will always lag behind industry. Once a particular steel has an acceptable history of performance, it may be incorporated into the applicable specifications. Until that time, the engineer must rely upon research data to determine the suitability of the part for a specific application.

A variety of tests have been devised over the years, each capable of measuring specific aspects of the weldability of the material under different conditions. Some tests measure the heat-affected zone properties, whereas others are more sensitive to weld-metal cracking tendencies. Unproven materials should be carefully reviewed by a competent engineer before being used in actual applications, and actual consideration of approximate weldability tests is recommended.

Listed in the following section are typical steels that are used for welded construction today.

3.1.2 Modern base metals for welding

The carbon steels. Classification of the carbon steels is based principally on carbon content. The groups are low carbon (to 0.30% carbon), medium carbon (0.30 to 0.45%), and high carbon (more than 0.45%). Mechanical properties of hot finished steels are influenced principally by chemical composition (particularly carbon content), but other factors—finishing temperature, section size, and the presence of residual elements—also affect properties. A ¾-in plate, for example, has higher tensile properties and lower elongation than a 1½-in plate of the same composition, resulting primarily from the higher rate of cooling of the ¾-in plate from the rolling temperature. Medium- and high-carbon steels are not typically used for structural operations and therefore will not be discussed further.

Low-carbon steels. In general, steels with carbon contents to 0.30% are readily joined by all the common arc-welding processes. These grades account for the greatest tonnage of steels used in welded structures. Typical applications include structural assemblies, as well as many other areas.

Steels with very low carbon content—to 0.13%—are generally good welding steels, but they are not the best for high-speed production welding. The low carbon content and the low manganese content (to 0.30%) tend to produce internal porosity. This condition is usually corrected by modifying the welding procedure slightly—usually by using a slower speed. Steels with very low carbon content are more ductile and easier to form than higher-carbon steels. They are used for applications requiring considerable cold forming, such as stampings or rolled or formed shapes.

Steels with 0.15 to 0.20% carbon content generally have excellent weldability, and they can be welded with all types of mild-steel electrodes. These steels should be used for maximum production speed on assemblies or structures that require extensive welding.

Steels at the upper end of the low carbon range—the 0.25 to 030% carbon grades—generally have good weldability, but when one or more of the elements is on the high side of permissible limits, cracking can result, particularly in fillet welds. With slightly reduced speeds and currents, any mild steel type of electrode can be used. In thicknesses up to $^{5}\!/_{16}$ in, standard procedures apply.

If some of the elements—particularly carbon, silicon, or sulfur—are on the high side of the limits, surface holes may form. Reducing current and speed minimizes this problem.

Although for some welding applications these steels require little or no preheating, heavy sections (2 in or more) and certain joint configurations often require a preheat. In general, steels in the 0.25 to 0.30% carbon range should be welded with low-hydrogen processes.

High-strength–low-alloy structural steels. Higher mechanical properties and better corrosion resistance than that of structural carbon steels are characteristics of the high-strength–low-alloy (HSLA) steels. These improved properties are achieved by addition of small amounts of alloying elements. Some of the HSLA types are carbon-manganese steels; others contain different alloy additions, governed by requirements for weldability, formability, toughness, or economy. The strength of these steels is generally between that of structural carbon steels and that of high-strength quenched and tempered steels.

High-strength–low-alloy steels are usually used in the as-rolled condition, although some are available that require heat treatment after fabrication. These steels are produced to specific mechanical property requirements rather than to chemical compositions.

Minimum mechanical properties available in the as-rolled condition vary among the grades and, within most grades, with thickness. Ranges of properties available in this group of steels are

1. Minimum yield point from 42,000 to 70,000 psi
2. Minimum tensile strength from 60,000 to 85,000 psi
3. Resistance to corrosion, classified as: equal to that of carbon steels, twice that of carbon steels, or 4 to 6 times that of carbon steels

The high-strength–low-alloy steels should not be confused with the high-strength quenched and tempered alloy steels. Both groups are sold primarily on a trade name basis, and they frequently share the same trade name, with different letters or numbers being used to identify each. The quenched and tempered steels are full-alloy steels that are heat-treated at the mill to develop optimum properties. They are generally martensitic in structure, whereas the HSLA steels are mainly ferritic steels; this is the clue to the metallurgical and fabricating differences between the two types. In the as-rolled condition, ferritic steels are composed of relatively soft, ductile constituents; martensitic steels have hard, brittle constituents that require heat treatment to produce their high-strength properties.

Strength in the HSLA steels is achieved instead by relatively small amounts of alloying elements dissolved in a ferritic structure. Carbon content rarely exceeds 0.28% and is usually between 0.15 and 0.22%. Manganese content ranges from 0.85 to 1.60%, depending on grade, and other alloy additions—chromium, nickel, silicon, phosphorus, copper, vanadium, columbium, and nitrogen—are used in amounts of less than 1%. Welding, forming, and machining characteristics of most grades do not differ markedly from those of the low-carbon steels.

To be weldable, the high-strength steels must have enough ductility to avoid cracking from rapid cooling. Weldable HSLA steels must be sufficiently low in carbon, manganese, and all "deep-hardening" elements to ensure that appreciable amounts of martensite are not formed upon rapid cooling. Superior strength is provided by solution of the alloying elements in the ferrite of the as-rolled steel. Corrosion resistance is also increased in certain of the HSLA steels by the alloying additions.

ASTM specifications. Thirteen ASTM specifications cover the plain-carbon, high-strength–low-alloy, and quenched and tempered structural steels. All of the following steels, except those noted, are prequalified according to D1.1-98:

ASTM A36: Covers carbon steel shapes, plates, and bars of structural quality for use in bolted or welded construction of bridges and buildings and for general structural purposes. Strength requirements are 58- to 80-ksi tensile strength and a minimum of 36-ksi yield strength. In general, A36 is weldable and is available in many shapes and sizes. However, its strength is limited only by a minimum value and could have yield strengths exceeding 50 ksi. Therefore, preheat and low-hydrogen electrodes may be required.

ASTM A53: Covers seamless and welded black and hot-dipped galvanized steel pipe. Strength requirements range from 25- to 35-ksi minimum yield and 45- to 60-ksi minimum tensile for three types and two grades. While maximum carbon levels are between 0.25 and 0.30%, pipe is typically manufactured by electric-resistance welding and submerged arc welding. A53 may require low-hydrogen process controls and preheat, especially when the carbon levels are close to the maximum allowed.

ASTM A441: Covers the intermediate-manganese HSLA steels that are readily weldable with proper procedures. The specification calls for additions of vanadium and a lower manganese content (1.25% maximum) than ASTM A440, which is generally not weldable due to its high carbon and manganese leads. Minimum mechanical properties vary from 42- to 50-ksi yield to 63- to 70-ksi tensile. Atmospheric corrosion resistance of this steel is approximately twice that of structural carbon steel, while it has superior toughness at low temperatures. Only shapes, plates, and bars are covered by the specification, but weldable sheets and strip can be supplied by some producers with approximately the same minimum mechanical properties.

ASTM A500: Covers cold-formed welded and seamless carbon steel round, square, rectangular, or special-shape structural tubing for welded, riveted, or bolted construction of bridges and buildings and for general structural purposes, and is commonly used in the United States. This tubing is produced in both seamless and welded sizes with a maximum periphery of 64 in and a maximum wall thickness of 0.625 in. Minimum strength properties range from 33-ksi yield and 45-ksi tensile for grade A to 46-ksi yield and 62-ksi tensile for grade C.

ASTM A501: Covers hot-formed welded and seamless carbon steel square, round, rectangular, or special-shape structural tubing for welded, riveted, or bolted construction of bridges and buildings and for general structural purposes, and is commonly used in Canada. Square and rectangular tubing may be furnished in sizes from 1 to 10 in across flat sides and wall thicknesses from 0.095 to 1.00 in. Minimum strength requirements are 36-ksi yield and 58-ksi tensile.

ASTM A514: Covers quenched and tempered alloy steel plates of structural quality in thicknesses of 6 in and under intended primarily for use in welded bridges and other structures. Strength requirements range from 100- to 130-ksi tensile and 90- to 100-ksi yield. When welding, the heat input must be controlled and specific minimum and maximum levels of heat input are required. Additionally, a low-hydrogen process is required.

ASTM A516: Covers carbon steel plates intended primarily for service in welded pressure vessels where improved notch toughness is important. These plates are furnished in four grades with strength requirements ranging from 55- to 90-ksi tensile and 30- to 38-ksi yield.

ASTM A572: Includes six grades of high-strength–low-alloy structural steels in shapes, plates, and bars used in buildings and bridges. These steels offer a choice of strength levels ranging from 42- to 65-ksi yields. Proprietary HSLA steels of this type with 70- and 75-ksi yield points are also available. Increasing care is required for welding these steels as the strength level increases.

A572 steels are distinguished from other HSLA steels by their columbium, vanadium, and nitrogen content.

A supplementary requirement is included in the specification that permits designating the specific alloying elements required in the steel. Examples are the type 1 designation, for columbium; type 2, for vanadium; type 3, for columbium and vanadium; and type 4, for vanadium and nitrogen. Specific grade designations must accompany this type of requirement.

ASTM A588: Covers high-strength–low-alloy structural steel shapes, plates, and bars for welded, riveted, or bolted connection. However, it is intended primarily for use in welded bridges and buildings in its unpainted condition, since the atmospheric corrosion resistance in most environments is substantially better than that of carbon steels. When properly exposed to the atmosphere, this steel can be used bare (unpainted) for many applications.

If the steel is to be painted, a low-hydrogen electrode without special corrosion resistance can be used. However, if the steel is to remain bare, then an electrode must be selected that has similar corrosion characteristics.

ASTM A709: Covers carbon and high-strength–low-alloy steel structural shapes, plates, and bars and quenched and tempered alloy steel for structural plates intended for use in bridges. Six grades are available in four yield strength levels of 36, 50, 70, and 100 ksi. Grades 50W, 70W, and 100W have enhanced atmospheric corrosion resistance. From a welding point of view, these grades are essentially the same as A36, A572, A852, and A514, respectively.

ASTM A710: Covers low-carbon age-hardening nickel-copper-chromium-molybdenum-columbium, nickel-copper-columbium, and nickel-copper-manganese-molybdenum-columbium alloy steel plates for general applications. Three different grades and three different conditions provide minimum yield strengths from 50 to 90 ksi. When this steel is to be welded, a welding procedure should be developed for the specific grade of steel and intended service. According to D1.1-98, no preheat is required with SMAW, SAW, GAW, and FCAW electrodes that are capable of depositing weld metal with a maximum diffusible hydrogen content of 8 mL/100 g.

ASTM A852: Covers quenched and tempered high-strength–low-alloy structural steel plates for welded, riveted, or bolted construction. It is intended primarily for use in welded bridges and buildings where savings in weight, added durability, and good notch toughness are important. This steel specification has substantially better atmospheric corrosion resistance than that of carbon structural steels. It has similar chemistry requirements to A588, but has been quenched and tempered to achieve the higher-strength level. Welding technique is important, and a welding procedure suitable for the steel and intended service should be developed. The specification limits the material thickness up to and including 4 in. According to D1.5-96, A852 is an approved bare metal under the A709 specification and D1.1-98 requires welding procedure qualification for this steel.

ASTM 913: Covers high-strength–low-alloy structural steel shapes in grades 60, 65, and 70 produced by the quenching and self-tempering process. The shapes are intended for riveted, bolted, or welded construction of bridges, buildings, and other structures. Although not in D1.5, the maximum yield strengths are 60, 65, and 70 ksi for the respective grades, while the minimum tensile strengths are 75, 80, and 90 ksi. A913 can be welded with a low-hydrogen process, and according to D1.1-98, it must provide a maximum diffusible hydrogen content of 8 mL/100 g. The shapes should not be formed or postweld heat-treated at temperatures exceeding 1100°F (600°C).

3.1.3 Older and miscellaneous base metals

Cast iron. Cast iron was a popular building material through the late 1800s, and occasionally an engineer is faced with the need to make additions to a cast-iron column, for example. Cast iron may also be encountered in miscellaneous structural applications such as ornate light poles, archways, and other components with decorative functions in addition to accomplishing structural support. Cast iron can be successfully welded but with great difficulty. Unless the welding involves repair of casting defects (voids, slag, or sand pockets), or the reattachment of nonstructural components, it is highly desirable to investigative alternative methods of joining when cast iron is involved. Bolted attachments are generally more easily made than welded ones.

Although there are several types of cast iron, the following are general guidelines to follow when welding cast iron. First, determine what type of cast iron is to be welded (that is, gray, malleable, ductile, or white). If the casting is white iron, which is generally considered unweldable, then an alternative jointing or repairing procedure should be investigated. However, if the casting is gray, malleable, or ductile, an appropriate welding procedure can be developed.

Second, preheating is *almost always* required when welding cast iron. Preheating can be achieved by heating large parts in a furnace or by using a heating torch on smaller parts. In general, the minimum preheat temperature will be around 500°F to sufficiently retard the cooling rate.

Third, any welding processes can be utilized; however, the electrode used should be a cast-iron electrode as specified in AWS A5.15.

Finally, after welding, the casting should be allowed to cool slowly to help reduce the hardness in the heat-affected zone. If the casting is a load-bearing member, caution must be taken to prohibit brittle fracture. The welding procedures should be tested, and a welding expert should be consulted.

Cast steels. Steel castings are often used for miscellaneous structural components such as rockers on expansion joints, miscellaneous brackets, and architectural elements. Welding cast steel is generally similar to welding of rolled steel of similar chemical composition. However, steel castings are often made of significantly different chemical compositions than would be utilized for rolled steel, particularly structural steels expected to be joined by welding. For example, a steel casting made of AISI 4140, a chromium-molybdenum steel with 0.40% carbon, will have reduced weldability because of the alloy content and the high carbon level. The producers of the steel castings often utilize alloys with reduced weldability simply because it may be easier to obtain the desired mechanical properties with the enriched composition, and welding of these materials is often overlooked.

Steel castings with poor weldability should be joined by bolting when possible, or alternative compositional requirements should be pursued. For example, it may be possible to select material with lower carbon and/or alloy levels for a specific application, increasing the weldability. If the chemistry of the casting is similar to that of a rolled steel, it will have similar weldability characteristics. However, the casting will have associated flow pattern–dependent properties.

Stainless steels. Because of its expense, stainless steel is rarely used for structural applications. The unique characteristics of stainless steel, however, make it ideally suited for applications where the structural material is subjected to corrosive environments, high or low service temperatures, and for applications where the material is to be used in its uncoated state. Certain grades of stainless steel are readily weldable, whereas others are welded with great difficulty.

When stainless steel is used as a structural material, particularly when it is joined to carbon steel elements, it is important to recognize the difference in thermal expansion between the two materials. With stainless expansion rates being 1.5 times that of carbon steel, the differential expansion can cause problems in structures that are subjected to variations in temperature.

The American Welding Society is currently developing a welding code to govern the fabrication of stainless-steel structures. It will be known as AWS D1.6 and, although not complete at the time of the writing of this chapter, it should be available in the near future. This code will provide welding requirements similar to those contained in AWS D1.1, but will deal specifically with stainless steel as the base material.

Metallurgy of stainless steel. Stainless steels are iron-chromium alloys, usually with a low carbon content, containing at least 11.5% chromium, which is the level at which effective resistance to atmospheric corrosion begins.

The American Iron and Steel Institute (AISI) classifies stainless steels by their metallurgical structures. This system is useful because the structure (austenitic, ferritic, or martensitic) indicates the general range of mechanical and physical properties, formability, weldability, and hardenability. The austenitic type generally have good weldability characteristics, high ductility, low yield strength, and high ultimate strength characteristics that make them suitable for forming and deep drawing operations; they also have the highest corrosion resistance of all the stainless steels. Austenitic grades account for the highest tonnage of weldable stainless steels produced.

Ferritic stainless steels are characterized by high levels of chromium and low carbon, plus additions of titanium and columbium. Since little or no austenite is present, these grades do not transform to martensite upon cooling, but remain ferritic throughout their normal operating temperature range. Principal applications of the ferritic types are automotive and appliance trim, chemical processing equipment, and products requiring resistance to corrosion and scaling at elevated temperatures, rather than high strength. Ferritic stainless steels are not easily welded in structural applications. If welding is required, a welding expert should be consulted for structural applications.

Martensitic stainless steels are iron-chromium alloys that can be heat treated to a wide range of hardness and strength levels. Martensitic grades are typically used to resist abrasion. They are not as corrosion resistant as the austenitic and ferritic types. Martensitic stainless steels, like the ferritic, are not easily welded in structural applications, and a welding expert should be consulted when welding.

Aluminum. Aluminum has many characteristics that are highly desirable for engineering applications, including structural, such as high strength-to-weight ratio, and corrosion resistance. Aluminum does not have the high modulus of elasticity associated with steel, but the weight-to-modulus ratio of the two materials is roughly equal. Aluminum is readily welded, but the welded connection rarely duplicates the strength of unwelded base metal. This is because the heat-affected zone (HAZ) in the as-welded state has reduced strength compared to the unaffected base material. This is in stark contrast to the behavior of steel, where the entire welded connection can usually be made as strong as the base material. The degree of strength degradation depends on the particular alloy system used. However, the engineer can conservatively assume that the heat-affected zone will have approximately one-half the strength of the aluminum alloy.

This characteristic is not necessarily a strong impediment to the use of aluminum, however. Creative joint designs and layouts of material can minimize the effect of the reduced strength HAZ. For example, rather than employing a butt joint perpendicular to the primary tensile loading, it may be possible to reorient the joint so that it lies parallel to the stress field, minimizing the magnitude of stress transfer across this interface, and thus reducing the effects of the reduced strength HAZ. Gussets, plates, stiffeners, and increases in thickness of the material at transition points can also be helpful in overcoming this characteristic.

Aluminum cannot be welded to steel or stainless steel by conventional arc-welding processes. It is possible to join aluminum to other materials by alternative welding processes such as explosion bonding. A common approach to welding aluminum to other significantly different materials is to utilize explosion bonding to create a transition member. In the final application, the steel, for example, is welded to the steel portion of the transition member, and the aluminum is welded to the aluminum side. While generally not justified for structural applications, this approach has been used for piping applications, for example. For structural applications, mechanical fasteners are generally employed; however, the galvanic action should be considered.

The requirements for fabrication of aluminum are contained in AWS D1.2 *Structural Welding Code—Aluminum.*

3.2 Weld Cracking/Solutions

Weld cracking is a problem faced occasionally by the fabricator. This section will discuss the various types of cracking and possible solutions for steel alloys.

Several types of discontinuities may occur in welds or heat-affected zones. Welds may contain porosity, slag inclusions, or cracks. Of the three, cracks are by far the most detrimental. Whereas there are acceptable limits for slag inclusions and porosity in welds, cracks are never acceptable. Cracks in, or in the vicinity of, a weld indicate that one or more problems exist that must be addressed. A careful analysis of crack characteristics will make it possible to determine their cause and take appropriate corrective measures.

For the purposes of this section, "cracking" will be distinguished from weld failure. Welds may fail due to overload, underdesign, or fatigue. The cracking discussed here is the result of solidification, cooling, and the stresses that develop due to weld shrinkage. *Weld cracking* occurs close to the time of fabrication. *Hot cracks* are those that occur at elevated temperatures and are usually solidification related. *Cold cracks* are those that occur after the weld metal has cooled to room temperature and may be hydrogen related. Neither is the result of service loads.

Most forms of cracking result from the shrinkage strains that occur as the weld metal cools. If the contraction is restricted, the strains will induce residual stresses that cause cracking. There are two opposing forces: the stresses induced by the shrinkage of the metal and the surrounding rigidity of the base material. The shrinkage stresses increase as the volume of shrinking metal increases. Large weld sizes and deep penetrating welding procedures increase the shrinkage strains. The stresses induced by these strains will increase when higher-strength filler metals and base materials are involved. With a higher yield strength, higher residual stresses will be presented.

Under conditions of high restraint, extra precautions must be utilized to overcome the cracking tendencies which are described in the following sections. It is essential to pay careful attention to welding sequence, preheat and interpass temperatures, postweld heat treatment, joint design, welding procedures, and filler material. The judicious use of peening as an in-process stress relief treatment may be necessary to fabricate highly restrained members.

3.2.1 Centerline cracking

Centerline cracking is characterized as a separation in the center of a given weld bead. If the weld bead happens to be in the center of the joint, as is always the case on a single-pass weld, centerline cracks will be in the center of the joint. In the case of multiple-pass welds, where several beads per layer may be applied, a centerline crack may not be in the geometric center of the joint, although it will always be in the center of the bead (Fig. 3.1).

Centerline cracking is the result of one of the following three phenomena: *segregation-induced cracking, bead shape–induced cracking,* or *surface profile–induced cracking*. Unfortunately, all three phenomena reveal themselves in the same type of crack, and it is often difficult to identify the cause. Moreover, experience has shown that often two or even all three of the phenomena will interact and contribute to the cracking problem. Understanding the fundamental mechanism of each of these types of centerline cracks will help in determining the corrective solutions.

Segregation-induced cracking occurs when low melting-point constituents, such as phosphorus, zinc, copper, and sulfur compounds, in the admixture separate during the weld solidification process. Low melting-point components in the molten metal will be forced to the center of the joint during solidification, since they are the last to solidify and the weld tends to separate as the solidified metal contracts away from the center region containing the low melting-point constituents.

When centerline cracking induced by segregation is experienced, several solutions may be implemented. Since the contaminant usually comes from the base material, the first consideration is to limit the amount of contaminant pickup from the base material. This may be done by limiting the penetration of the welding process. In some cases, a joint redesign may be desirable. The extra penetration afforded by some of the processes is not necessary and can be reduced. This can be accomplished by using lower welding currents.

A buttering layer of weld material (Fig. 3.2) deposited by a low-energy process, such as shielded metal arc welding, may effectively reduce the amount of pickup of contaminant into the weld admixture.

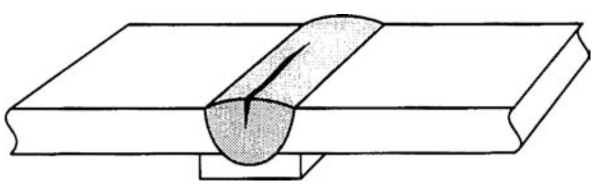


Figure 3.1 Centerline cracking. (*Courtesy of The Lincoln Electric Company.*)

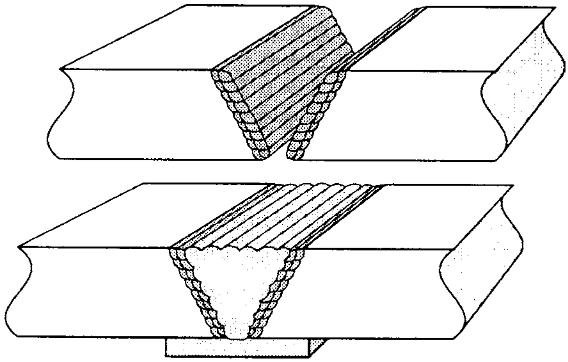


Figure 3.2 Buttering. (*Courtesy of The Lincoln Electric Company.*)

In the case of sulfur, it is possible to overcome the harmful effects of iron sulfides by preferentially forming manganese sulfide. Manganese sulfide (MnS) is created when manganese is present in sufficient quantities to counteract the sulfur. Manganese sulfide has a melting point of 2900°F. In this situation, before the weld metal begins to solidify, manganese sulfides are formed which do not segregate. Steel producers utilize this concept when higher levels of sulfur are encountered in the iron ore. In welding, it is possible to use filler materials with higher levels of manganese to overcome the formation of low melting-point iron sulfide. Unfortunately, this concept cannot be applied to contaminants other than sulfur.

The second type of centerline cracking is known as *bead shape–induced cracking*. This is illustrated in Fig. 3.3 and is associated with deep penetrating processes such as SAW and CO_2-shielded FCAW. When a weld bead is of a shape where there is more depth than width to the weld cross section, the solidifying grains growing perpendicular to the steel surface intersect in the middle, but do not gain fusion across the joint. To correct for this condition, the individual weld beads must have at least as much width as depth. Recommendations vary

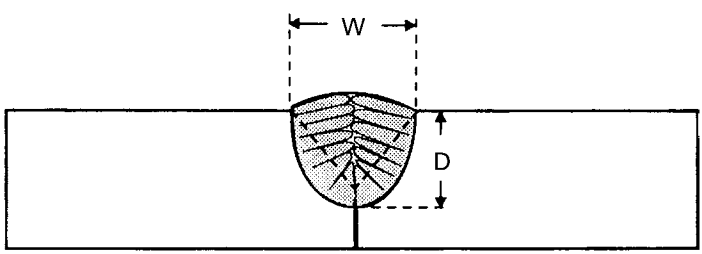


Figure 3.3 Bead shape–induced cracking. (*Courtesy of The Lincoln Electric Company.*)

from a 1:1 to a 1.4:1 width-to-depth ratio to remedy this condition. The total weld configuration, which may have many individual weld beads, can have an overall profile that constitutes more depth than width. If multiple passes are used in this situation, and each bead is wider than it is deep, a crack-free weld can be made.

When centerline cracking due to bead shape is experienced, the obvious solution is to change the width-to-depth relationship. This may involve a change in joint design. Since the depth is a function of penetration, it is advisable to reduce the amount of penetration. This can be accomplished by utilizing lower welding amperages and larger-diameter electrodes. All of these approaches will reduce the current density and limit the amount of penetration.

The final mechanism that generates centerline cracks is *surface profile–induced conditions.* When concave weld surfaces are created, internal shrinkage stresses will place the weld metal on the surface into tension. Conversely, when convex weld surfaces are created, the internal shrinkage forces pull the surface into compression. These situations are illustrated in Fig. 3.4. Concave weld surfaces frequently are the result of high arc voltages. A slight decrease in arc voltage will cause the weld bead to return to a slightly convex profile and eliminate the cracking tendency. High travel speeds may also result in this configuration. A reduction in travel speed will increase the amount of fill and return the surface to a convex profile. Vertical-down welding also has a tendency to generate these crack-sensitive concave surfaces. Vertical-up welding can remedy this situation by providing a more convex bead.

3.2.2 Heat-affected zone cracking

Heat-affected zone (HAZ) cracking (Fig. 3.5) is characterized by separation that occurs immediately adjacent to the weld bead. Although it

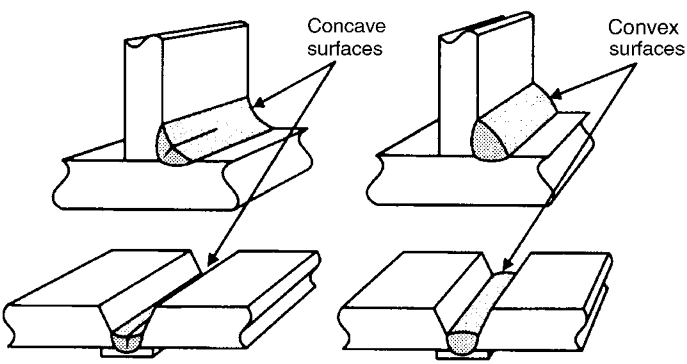


Figure 3.4 Surface profile–induced cracking. (*Courtesy of The Lincoln Electric Company.*)

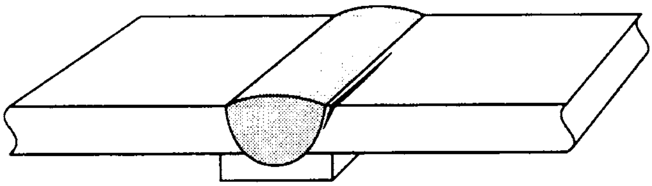


Figure 3.5 Heat-affected zone cracking. (*Courtesy of The Lincoln Electric Company.*)

is related to the welding process, the crack occurs in the base material, not in the weld material. This type of cracking is also known as *underbead cracking, toe cracking,* or *delayed cracking*. Because this cracking occurs after the steel has cooled below approximately 200°C, it can be called *cold cracking,* and because this cracking is associated with hydrogen, it is also called *hydrogen-assisted cracking*.

In order for heat-affected zone cracking to occur, three conditions must be present simultaneously: (1) there must be a sufficient level of hydrogen, (2) there must be a sufficiently sensitive material involved, and (3) there must be a sufficiently high level of residual or applied stress. Sufficient reduction or elimination of one of the three variables will eliminate heat-affected zone cracking. In welding applications, the typical approach is to limit two of the three variables, namely, the level of hydrogen and the sensitivity of the material. Hydrogen can enter into a weld pool through a variety of sources. Moisture and organic compounds are the primary sources of hydrogen. It may be present on the steel, electrode, in the shielding materials, and in atmospheric humidity. Flux ingredients, whether on the outside of electrodes, inside the core of electrodes, or in the form of submerged arc or electroslag fluxes, can adsorb or absorb moisture, depending on storage conditions and handling practices. To limit hydrogen content in deposited welds, welding consumables must be properly maintained, and welding must be performed on surfaces that are clean and dry. The second necessary condition for heat-affected zone cracking is a sensitive microstructure. In the case of heat-affected zone cracking, the area of interest is the heat-affected zone that results from the thermal cycle experienced by the region immediately surrounding the weld nugget. As this area is heated by the welding arc during the creation of the weld pool, it transforms from its room temperature structure of ferrite to the elevated temperature structure of austenite. The subsequent cooling rate will determine the resultant HAZ properties. Conditions that encourage the development of crack-sensitive microstructures include high cooling rates and higher hardenability levels in the steel. High cooling rates are encouraged by lower heat-input welding procedures, greater base material thicknesses, and colder base metal temperatures. Higher hardenability levels result from higher carbon contents and/or alloy levels. For a given steel, the most effective way to reduce the cooling rate is by raising the temperature of the surrounding steel through preheat. This reduces the temperature gradient, slowing cooling rates and limiting the formation of sensitive microstructures. Effective preheat is the primary means by which acceptable heat-affected zone properties are created, although heat input also has a significant effect on cooling rates in this zone.

The residual stresses of welding can be reduced through thermal stress relief, although for most structural applications, this is economically impractical. For complex structural applications, temporary shoring and other conditions must be considered as the steel will have a greatly reduced strength capacity at stress-relieving temperatures. For practical applications, heat-affected zone cracking will be controlled by effective low-hydrogen practice and appropriate preheats.

When sufficient levels of hydrogen, residual stress, and material sensitivity occur, hydrogen cracking will occur in the heat-affected zone. For this cracking to occur, it is necessary for the hydrogen to migrate into the heat-affected zone, an activity that takes time. For this reason, the D1.1 code requires a delay of 48 h for the inspection of welds made on A514 steel, known to be sensitive to hydrogen-assisted heat-affected zone cracking.

With time, hydrogen diffuses from weld deposits. Sufficient diffusion to avoid cracking normally takes place in a few weeks, although it may take many months depending on the specific application. The concentrations of hydrogen near the time of welding are always the greatest, and if hydrogen-induced cracking is to occur, it will generally occur within a few days of fabrication. However, it may take longer for the cracks to grow to a sufficient size to be detected.

Although a function of many variables, general diffusion rates can be approximated. At 450°F, hydrogen diffuses at the rate of approximately 1 in/h. At 220°F, hydrogen diffuses the same 1 in in approximately 48 h. At room temperature, typical diffusible hydrogen rates are 1 in/2 weeks. If there is a question regarding the level of hydrogen in a weldment, it is possible to apply a postweld heat treatment commonly called *postheat.* This generally involves the heating of the weld to a temperature of 400 to 500°F, holding the steel at that temperature for approximately 1 h for each inch of thickness of material involved. At that temperature, the hydrogen is likely to be redistributed through diffusion to preclude further risk of cracking. Some materials, however, will require significantly longer than 1 h/in. This operation is not necessary where hydrogen has been properly controlled, and it is not as powerful as preheat in terms of its ability to prevent underbead cracking. In order for postheat operations to be effective, they must be applied before the weldment is allowed to cool to room temperature. Failure to do so could result in heat-affected zone cracking prior to the application of the postheat treatment.

3.2.3 Transverse cracking

Transverse cracking, also called *cross-cracking,* is characterized as a crack within the weld metal perpendicular to the longitudinal direction (Fig. 3.6). This is the least frequently encountered type of cracking, and is generally associated with weld metal that is higher in strength, significantly overmatching the base material. Transverse cracking is also hydrogen assisted, and like heat-affected zone cracking, is also a factor of excessive hydrogen, residual stresses, and a sensitive microstructure. The primary difference is that transverse cracking occurs in the weld metal as a result of the longitudinal residual stress.

As the weld bead shrinks longitudinally, the surrounding base material resists this force by going into compression. The high strength of the surrounding steel in compression restricts the required shrinkage of the weld material. Due to the restraint of the surrounding base material, the weld metal develops longitudinal stresses which may facilitate cracking in the transverse direction.

When transverse cracking is encountered, a review of the low-hydrogen practice is warranted. Electrode storage conditions should be carefully reviewed. If these are proper, a reduction in the strength of the weld metal will usually solve transverse cracking problems. Of course, design requirements must still be met, although most transverse cracking results from weld metal overmatch conditions.

Emphasis is placed upon the weld metal because the filler metal may deposit lower-strength, highly ductile metal under normal conditions. However, with the influence of alloy pickup, it is possible for the weld metal to exhibit extremely high strengths with reduced ductility. Using lower-strength weld metal is an effective solution, but caution should be taken to ensure that the required joint strength is attained.

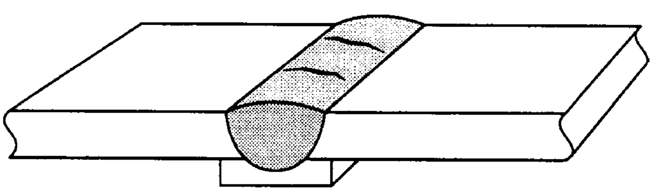


Figure 3.6 Transverse cracking. (*Courtesy of The Lincoln Electric Company.*)

Preheat may have to be applied to alleviate transverse cracking. The preheat will assist in diffusing hydrogen. As preheat is applied, it will additionally expand the length of the weld joint, allowing the weld metal and the joint to contract simultaneously, and reducing the applied stress to the shrinking weld. This is particularly important when making circumferential welds. When the circumference of the materials being welded is expanded, the weld metal is free to contract along with the surrounding base material, reducing the longitudinal shrinkage stress. Finally, postweld hydrogen-release treatments that involve holding the steel at 250 to 450°F for extended times will assist in diffusing any residual hydrogen.

3.3 Welding Processes

A variety of welding processes can be used for fabrication in structural applications. However, it is important that all parties involved understand these processes in order to ensure quality and economical fabrication. A brief description of the major processes is provided below.

3.3.1 SMAW

Shielded metal arc welding (SMAW), commonly known as *stick electrode welding* or *manual welding,* is the oldest of the arc-welding processes (Fig. 3.7). It is characterized by versatility, simplicity, and flexibility. The SMAW process is commonly used for tack welding, fabrication of miscellaneous components, and repair welding. There is a practical limit to the amount of current that may be used. The covered electrodes are typically 9 to 18 in long, and if the current is

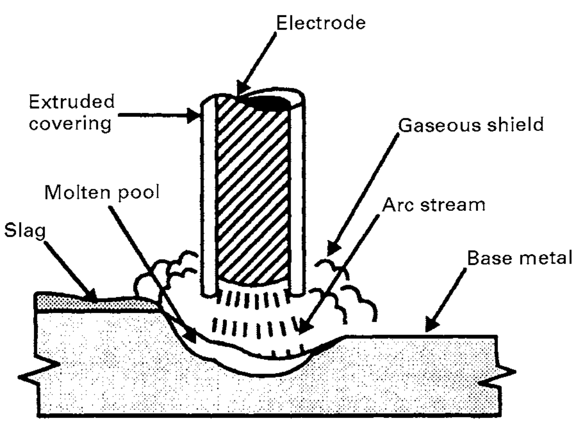


Figure 3.7 SMAW process. (*Courtesy of The Lincoln Electric Company.*)

raised too high, electrical-resistance heating within the unused length of electrode will become so great that the coating ingredients may overheat and "break down," resulting in potential weld quality degradation. SMAW also is used in the field for erection, maintenance, and repairs. SMAW has earned a reputation for depositing high-quality welds dependably. It is, however, slower and more costly than other methods of welding, and is more dependent on operator skill. Consequently, SMAW seldom is used for primary fabrication of structures.

3.3.2 FCAW

Flux-cored arc welding (FCAW) uses an arc between a continuous filler metal electrode and the weld pool. The electrode is always tubular. Inside the metal sheath is a combination of materials that may include metallic powder and flux. FCAW may be applied automatically or semiautomatically.

The flux-cored arc-welding process has become the most popular semiautomatic process for structural steel fabrication and erection. Production welds that are short, change direction, difficult to access, must be done out-of-position (that is, vertical or overhead), or part of a short production run generally will be made with semiautomatic FCAW.

The flux-cored arc-welding process offers two distinct advantages over shielded metal arc welding. First, the electrode is continuous. This eliminates the built-in starts and stops that are inevitable with shielded metal arc welding. Not only does this have an economic advantage because the operating factor is raised, but the number of arc starts and stops, a potential source of weld discontinuities, is reduced.

Another major advantage is that increased amperages can be used with flux-cored arc welding, with a corresponding increase in deposition rate and productivity. With the continuous flux-cored electrodes, the tubular electrode is passed through a contact tip, where electrical energy is transferred to the electrode. The short distance from the contact tip to the end of the electrode, known as *electrode extension* or *electrical stickout,* limits the buildup of heat due to electrical resistance. This electrode extension distance is typically $\frac{3}{4}$ to 1 in for flux-cored electrodes.

Within the category of flux-cored arc welding, there are two specific subsets: self-shielded flux core (FCAW-ss) (Fig. 3.8) and gas-shielded flux core (FCAW-g) (Fig. 3.9). Self-shielded flux-cored electrodes require no external shielding gas. The entire shielding system results from the flux ingredients contained within the core of the tubular electrode. The gas-shielded versions of flux-cored electrodes utilize an externally

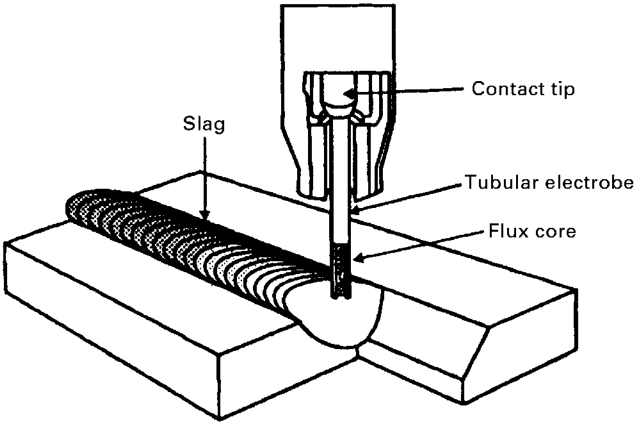


Figure 3.8 Self-shielded FCAW. (*Courtesy of The Lincoln Electric Company.*)

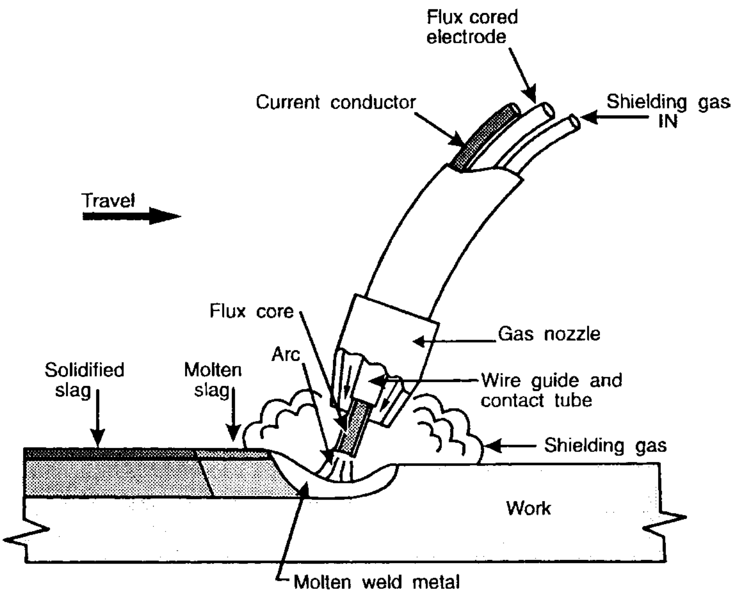


Figure 3.9 Gas-shielded FCAW. (*Courtesy of The Lincoln Electric Company.*)

supplied shielding gas. In many cases, CO_2 is used, although other gas mixtures may be used, for example, argon/CO_2 mixtures. Both types of flux-cored arc welding are capable of delivering weld deposits that meet the quality and mechanical property requirements for most structure applications. In general, the fabricator will utilize the process that offers the greatest advantages for the particular environment. Self-shielded flux-cored electrodes are better for field-welding situations. Since no externally supplied shielding gas is required, the process may be used in high winds without adversely affecting the quality of the deposit. With any of the gas-shielded processes, wind shields must be erected to preclude interference with the gas shield in windy weather. Many fabricators have found self-shielded flux core offers advantages for shop welding as well, since it permits the use of better ventilation.

Individual gas-shielded flux-cored electrodes tend to be more versatile than self-shielded flux-cored electrodes, and in general, provide better arc action. Operator appeal is usually higher. While the gas shield must be protected from winds and drafts, this is not particularly difficult in shop fabrication situations. Weld appearance and quality are very good. Higher-strength gas-shielded FCAW electrodes are available, while current technology limits self-shielded FCAW deposits to 90-ksi tensile strength or less.

3.3.3 SAW

Submerged arc welding (SAW) differs from other arc-welding processes in that a layer of granular material called *flux* is used for shielding the arc and the molten metal (Fig. 3.10). The arc is struck between

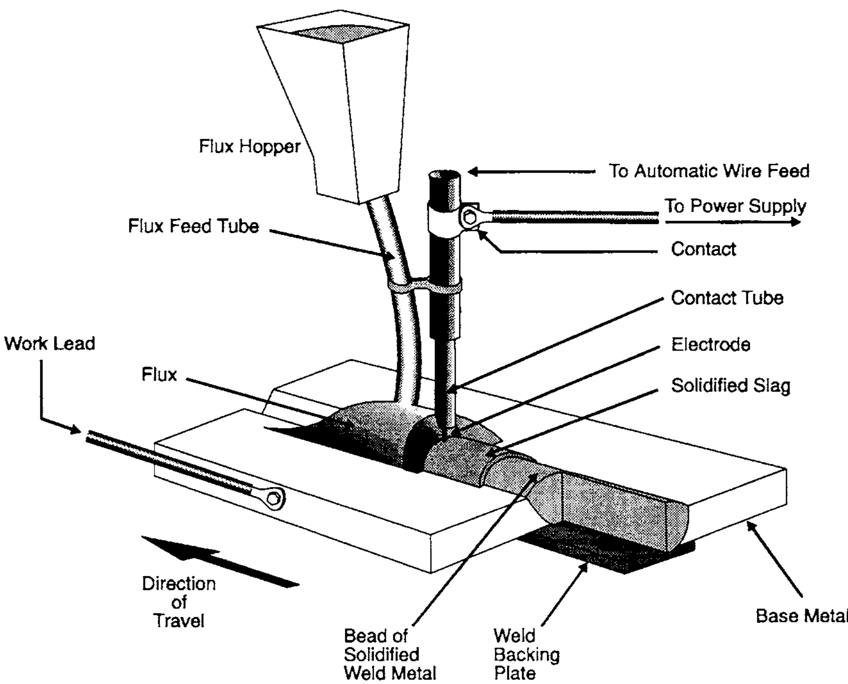


Figure 3.10 SAW process. (*Courtesy of The Lincoln Electric Company.*)

the workpiece and a bare wire electrode, the tip of which is submerged in the flux. Since the arc is completely covered by the flux, it is not visible and the weld is made without the flash, spatter, and sparks that characterize the open-arc processes. The nature of the flux is such that very little smoke or visible fumes are developed.

The process is typically fully mechanized, although semiautomatic operation is often utilized. The electrode is fed mechanically to the welding gun, head, or heads. In semiautomatic welding, the welder moves the gun, usually equipped with a flux-feeding device, along the joint. High currents can be used in submerged arc welding and extremely high-heat input levels can be developed. Because the current is applied to the electrode a short distance above its arc, relatively high amperages can be used on small-diameter electrodes, resulting in extremely high current densities. This allows for high deposition rates and deep penetration.

Welds made under the protective layer of flux (Fig. 3.10) are excellent in appearance and spatter-free. The high quality of submerged arc welds, the high deposition rates, the deep penetration characteristics, and the easy adaptability of the process to full mechanization make it popular for the manufacture of plate girders and fabricated columns.

One of the greatest benefits of the SAW process is freedom from the open arc. This allows multiple arcs to be operated in a tight, confined area without the need for extensive shields to guard the operators from arc flash. Yet this advantage also proves to be one of the chief drawbacks of the process; it does not allow the operator to observe the weld puddle. When SAW is applied semiautomatically, the operator must learn to propel the gun carefully in a fashion to ensure uniform bead contour. The experienced operator relies on the uniform formation of a slag blanket to indicate the nature of the deposit. For single-pass welds, this is mastered fairly readily; however, for multiple-pass welding, the skills required are significant. Therefore, most submerged arc applications are mechanized. The nature of the joint must then lend itself to automation if the process is to prove viable. Long, uninterrupted straight seams are ideal applications for submerged arc. Short, intermittent welds are better made with one of the open-arc processes.

Two electrodes may be fed through a single electrical contact tip, resulting in higher deposition rates. Generally known as *parallel electrode welding,* the equipment is essentially the same as that used for single-electrode welding, and parallel electrode welding procedures may be prequalified under AWS D1.1-98.

Multiple-electrode SAW refers to a variation of submerged arc which utilizes at least two separate power supplies, two separate wire drives, and feeds two electrodes independently. Some applications, such as the manufacture of line pipe, may use up to five independent electrodes in a multiple-electrode configuration. AC welding is typically used for multielectrode welding. If dc current is used, it is limited usually to the lead electrode to minimize the potentially negative interaction of magnetic fields between the two electrodes.

3.3.4 GMAW

Gas metal arc welding (GMAW) (Fig. 3.11) utilizes equipment similar to that used in flux-cored arc welding. Indeed, the two processes are very similar. The major differences are: gas metal arc uses a solid or metal-cored electrode and leaves no appreciable amount of residual slag.

Gas metal arc has not been a popular method of welding in the typical structural steel fabrication shop because of its sensitivity to mill scale, rust, limited puddle control, and sensitivity to shielding loss. Newer GMAW metal-cored electrodes, however, are beginning to be used in the shop fabrication of structural elements with good success.

A variety of shielding gases or gas mixtures may be used for GMAW. Carbon dioxide (CO_2) is the lowest-cost gas, and while acceptable for welding carbon steel, the gas is not inert but active at elevated temperatures. This has given rise to the term *MAG* (*metal active gas*) for the process when CO_2 is used, and *MIG* (*metal inert gas*) when predominantly argon-based mixtures are used. While

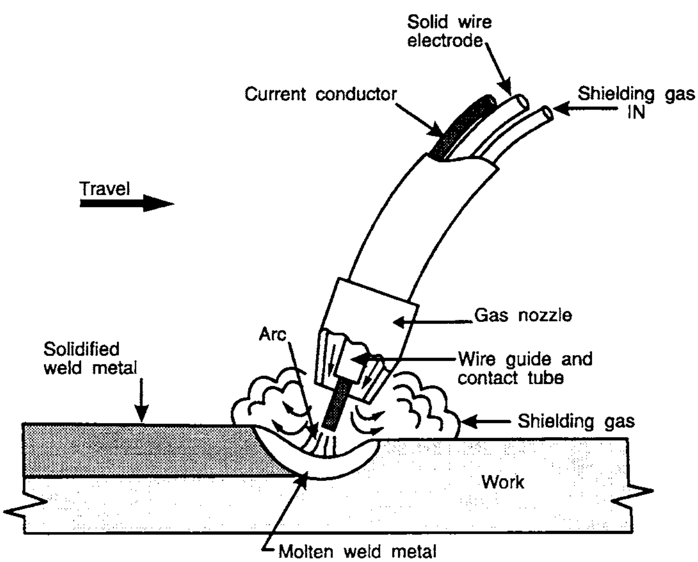


Figure 3.11 GMAW process. (*Courtesy of The Lincoln Electric Company.*)

shielding gas is used to displace atmospheric oxygen, it is possible to add smaller quantities of oxygen into mixtures of argon—generally at levels of 2 to 8%. This helps stabilize the arc and decreases puddle surface tension, resulting in improved wetting. Tri and quad mixes of argon, oxygen, carbon dioxide, and helium are possible, offering advantages that positively affect arc action, deposition appearance, and fume generation rates.

Short arc transfer is ideal for welding on thin-gauge materials. It is generally not suitable for structural steel fabrication purposes. In this mode of transfer, the small-diameter electrode, typically 0.035 or 0.045 in, is fed at a moderate wire feed speed at relatively low voltages. The electrode will touch the workpiece, resulting in a short in the electrical circuit. The arc will actually go out at this point, and very high currents will flow through the electrode, causing it to heat and melt. Just as excessive current flowing through a fuse causes it to blow, so the shorted electrode will separate from the work, initiating a momentary arc. A small amount of metal will be transferred to the work at this time.

The cycle will repeat itself again once the electrode shorts to the work. This occurs somewhere between 60 and 200 times/s, creating a characteristic buzz to the arc. This mode of transfer is ideal for sheet metal, but results in significant fusion problems if applied to heavy materials. A phenomenon known as *cold lap* or *cold casting* may result where the metal does not fuse to the base material. This is unacceptable since the welded connections will have virtually no strength. Great caution must be exercised in the application of the short arc mode to heavy plates. The use of short arc on heavy plates is not totally prohibited, however, since it is the only mode of transfer that can be used out-of-position with gas metal arc welding, unless specialized equipment is used. Weld joint details must be carefully designed when short arc transfer is used. Welders must pass specific qualification tests before using this mode of transfer. Short arc transfer is often abbreviated as GMAW-s, and is not prequalified by the D1.1 code.

Globular transfer is a mode of gas metal arc welding that results when high concentrations of carbon dioxide are used, resulting in an arc that is rough with larger globs of metal ejected from the end of the electrode. This mode of transfer, while resulting in deep penetration, generates relatively high levels of spatter. Weld appearance can be poor and it is restricted to the flat and horizontal position. Globular transfer may be preferred over spray arc transfer because of the low cost of CO_2-shielding gas and the lower level of heat experienced by the operator.

Spray arc transfer is characterized by high wire-feed speeds at relatively high voltages. A fine spray of molten drops, all smaller in diameter than the electrode diameter, is ejected from the electrode toward the work. Unlike short arc transfer, the arc in spray transfer is continuously maintained. High-quality welds with particularly good appearance are the result. The shielding used for spray arc transfer is composed of at least 80% argon, with the balance made up of either carbon dioxide or oxygen. Typical mixtures would include 90-10 argon-CO_2, and 95-5 argon-oxygen. Other proprietary mixtures are available from gas suppliers. Relatively high arc voltages are used with the spray mode of transfer. However, due to the intensity of the arc, spray arc is restricted to applications in the flat and horizontal position, because of the puddle fluidity and lack of a slag to hold the molten metal in place.

Pulsed arc transfer utilizes a background current that is continuously applied to the electrode. A pulsing peak current is optimally applied as a function of the wire-feed speed. With this mode of transfer, the power supply delivers a pulse of current which, ideally, ejects a single droplet of metal from the electrode. The power supply returns to a lower background current which maintains the arc. This occurs between 100 and 400 times/s. One advantage of pulsed arc transfer is that it can be used out-of-position. For flat and horizontal work, it may not be as fast as spray transfer. However, used out-of-position, it is free of the problems associated with the gas metal arc short-circuiting mode. Weld appearance is good and quality can be excellent. The disadvantage of pulsed arc transfer is that the equipment is slightly more complex and more costly. The joints are still required to be relatively clean, and out-of-position welding is still more difficult than with processes that generate a slag that can support the molten puddle.

Metal-cored electrodes are a relatively new development in gas metal arc welding. This is similar to flux-cored arc welding in that the electrode is tubular, but the core material does not contain slag-forming ingredients. Rather, a variety of metallic powders is contained in the core. The resulting weld is virtually slag-free, just as with other forms of GMAW. The use of metal-cored electrodes offers many fabrication advantages. They have increased ability to handle mill scale and other surface contaminants.

Finally, metal-cored electrodes permit the use of high amperages that may not be practical with solid electrodes, resulting in potentially higher deposition rates. The properties obtained from metal-cored deposits can be excellent. Appearance is very good. Because of the ability of the filler metal manufacturer to control the composition of the core ingredients, mechanical properties obtained from metal-cored deposits may be more consistent than those obtained with solid electrodes. However, metal-cored electrodes are, in general, more expensive.

3.3.5 ESW/EGW

Electroslag and electrogas welding (ESW/EGW) are closely related processes (Figs. 3.12 and 3.13) that offer high-deposition welding in the vertical plane. Properly applied, these processes offer significant savings over alternative out-of-position methods and, in many cases, savings over flat position welding. Although the two processes have similar applications and mechanical setup, there are fundamental differences in the arc characteristics. Electroslag and electrogas are

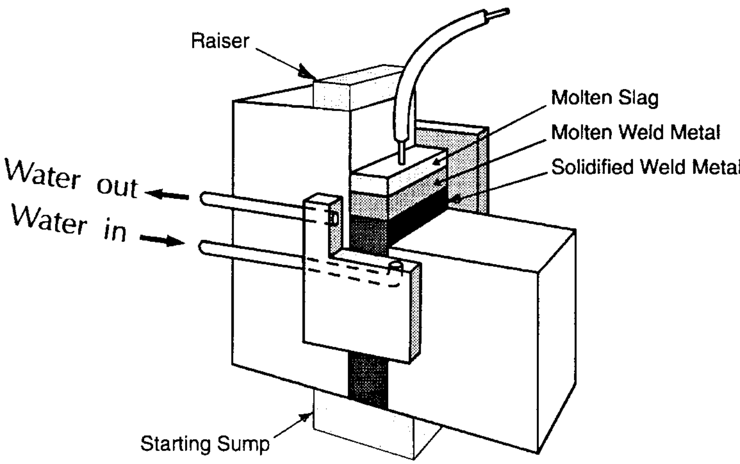


Figure 3.12 ESW process. (*Courtesy of The Lincoln Electric Company.*)

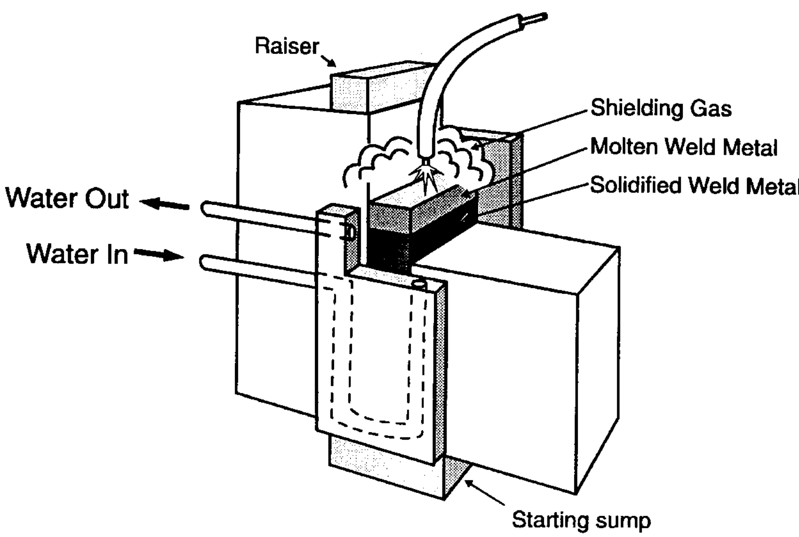


Figure 3.13 EGW process. (*Courtesy of The Lincoln Electric Company.*)

mechanically similar in that both utilize copper dams or shoes that are applied to either side of a square-edged butt joint. An electrode or multiple electrodes are fed into the joint. A starting sump is typically applied for the beginning of the weld. As the electrode is fed into the joint, a puddle is established that progresses vertically. The copper dams, which are commonly water-cooled, chill the weld metal and prevent it from escaping from the joint. The weld is completed in one pass.

These processes may be used for groove welds in butt, corner, and tee joints. Typical applications involve heavier plate, usually 1 in or thicker. Multiple electrodes may be used in a single joint, allowing very heavy plate up to several inches thick to be joined in a single pass. Because of the sensitivity of the process to the number of variables involved, specific operator training is required, and the D1.1-98 code requires welding procedures to be qualified by test.

In building construction, applications for ESW/EGW with traditional connection designs are somewhat limited. However, they can be highly efficient in the manufacture of tree columns. In the shop, the beam flange-to-column welds can be made with the column in the horizontal plane. With the proper equipment and tooling, all four flange welds can be made simultaneously. In addition, continuity plate welds can be made with ESW/EGW. Future connection designs may utilize configurations that are more conducive to these processes.

Another common application is for the welding of continuity plates inside box columns. It is possible to weld three sides of the continuity plate to the interior of the box prior to closing the box with the fourth side. However, once this closure is made, access to the final side of the continuity plate is restricted. This final closure weld can be made by operating through a hole in the outside of the box column. This approach is very popular in the Far East where box columns are widely used.

In electroslag welding, a granular flux is metered into the joint during the welding operation. At the beginning, an arc, similar to that of submerged arc welding, is established between the electrode and the sump.

After the initial flux is melted into a molten slag, the reaction changes. The slag, which is carefully designed to be electrically conductive, will conduct the welding current from the electrode through the slag into the pieces of steel to be joined. As high currents are passed through the slag, it becomes very hot. The electrode is fed through the hot slag and melts. Technically, electroslag welding is not an *arc*-welding process, but a *resistance*-welding process. Once the arc is extinguished and the resistance-melting process is stabilized, the weld continues vertically to completion. A small amount of slag is consumed as it chills against the water-cooled copper shoes. In some cases, steel dams instead of copper dams are used to retain the puddle. After completion of the weld, the steel dams stay in place, and become part of the final product. Slag must be replenished, and additional flux is continuously added to compensate for the loss.

One aspect of electroslag welding that must be considered is the very high heat input associated with the process. This causes a large heat-affected zone (HAZ) that may have a lower notch toughness. Electroslag welding is different from electroslag, inasmuch as no flux is used. Electrogas welding is a true arc-welding process and is conceptually more like gas metal arc or flux-cored arc welding. A solid or tubular electrode is fed into the joint, which is flooded with an inert gas shield. The arc progresses vertically while the puddle is retained by the water-cooled dams.

The HAZ performance is dependent not only on the heat input, but also on the nature of the steel. While all processes develop a heat-affected zone, the large size of the electroslag heat-affected zone justifies additional scrutiny. Advances in steel technology have resulted in improved steels, featuring higher cleanliness and toughness, that better retain the HAZ properties in ESW/EGW welds.

3.3.6 GTAW

The gas-tungsten arc-welding (GTAW) *process,* colloquially called *TIG welding,* is rarely used in structural applications. However, it may be specified to meet some unique requirements or for a repair welding procedure. GTAW (Fig. 3.14) uses a nonconsumed electrode composed

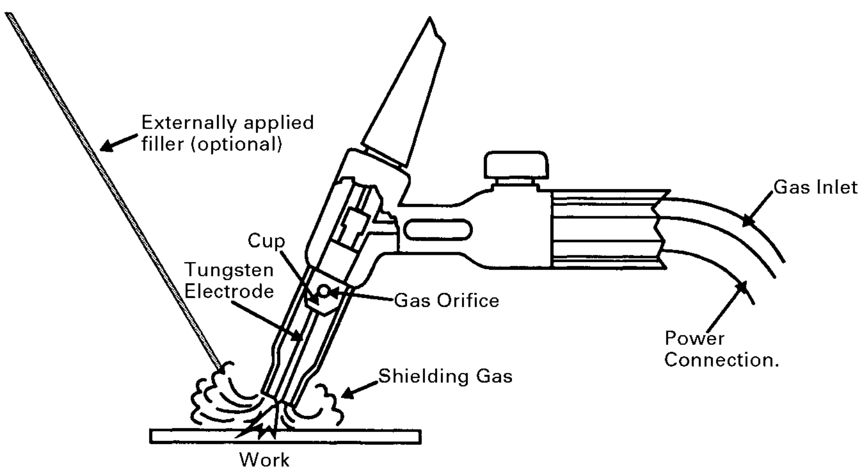


Figure 3.14 Gas-tungsten arc welding. (*Courtesy of The Lincoln Electric Company.*)

of tungsten, a metal with a very high melting point. Between the tungsten and the work, an arc is established that results in heating of the base material. A filler rod may or may not be used. The area is shielded with an inert gas, typically argon, although helium may be used. GTAW is ideally suited for welding on nonferrous materials, such as stainless steel and aluminum. Moreover, it is very effective when joining thin sections.

One area where gas-tungsten arc welding may be used in structural applications is when it is applied for the purpose of "TIG dressing." The TIG dressing technique has been used to extend the fatigue life of fillet welds. With this technique, the gas-tungsten arc process is used to heat and melt the toes of fillet welds, resulting in a new distribution of residual stresses and perhaps improved contour of the toe of the fillet. This has been used to retrofit structures where fatigue cracking is expected. The process is inherently expensive, but may be justified if it extends the life of the structure.

3.4 Welding Process Selection

Any of the common arc-welding processes can be used to achieve the quality required for structural steel applications. While each may have a particular area of strength and/or weakness, the primary consideration as to which process will be used is largely cost-driven. The availability of specialized equipment in one fabrication shop compared to the capabilities of a second shop may dictate significantly different approaches, both of which may prove to be cost-effective. A history of successful usage offers a strong incentive for the fabricator to continue using a given process. The reasons for this go well beyond familiarity and comfort with a specific approach. When welders and procedures are established with a given process, significant costs will be incurred with any change to a new approach.

3.4.1 Joint requirements

Each individual weld-joint configuration and preparation has certain process requirements in order to achieve low-cost welding. Four characteristics must be considered: deposition rate, penetration ability, out-of-position capability, and high travel-speed capacity. Each process exhibits different capabilities in these realms. Once the joint and its associated requirements are analyzed, they should be compared to the various process options and the ability of the process to achieve those requirements. A proper match of weld-joint requirements and process capabilities will lead to dependable and economical fabrication.

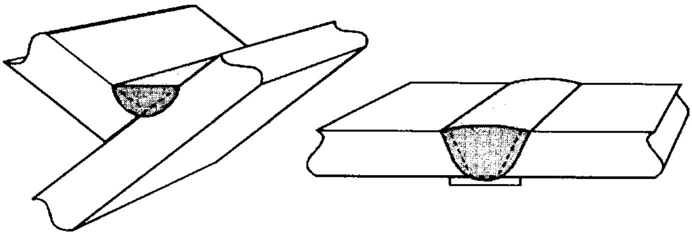


Figure 3.15 Joint requiring substantial fill. (*Courtesy of The Lincoln Electric Company.*)

Some welds, such as large fillet welds and groove welds, require that *high deposition-rate* welding be used (Fig. 3.15) for the most economical fabrication. The cost of making these welds will be determined largely by the deposition rate of the process. The amount of weld material required may be measured in pounds per foot of joint. Once the deposition rate of a process in pounds per hour is known, it is possible to determine the number of feet of weld that can be made in a given hour assuming 100% arc time. This, of course, translates directly to productivity rates.

The second criterion imposed by weld joints is the requirement for *penetration*. Examples are listed under Fig. 3.16 and would include any complete joint-penetration groove weld that has a root face dimension. These joints will be made by welding from one side and back-gouging from the other to ensure complete fusion. With deeper penetration afforded by the welding process, a smaller amount of base metal will have to be removed by back-gouging. Subsequent welding will then be proportionately reduced as well.

While all welding requires fusion, not all joints require deep penetration. For example, simple fillet welds are required by AWS D1.1-98 to have fusion to the root of the joint, but are not required to have penetration beyond the root. This has a practical basis: verification of

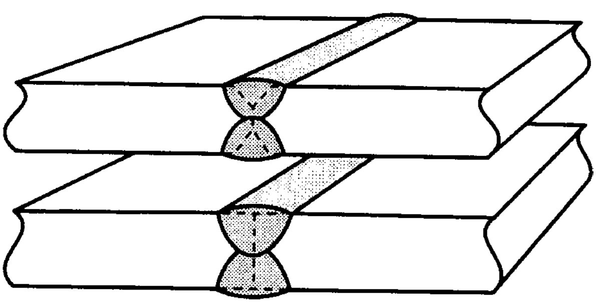


Figure 3.16 Joints requiring substantial penetration. (*Courtesy of The Lincoln Electric Company.*)

penetration beyond the root is impossible with visual inspection. Fusion to the root, and not necessarily beyond, ensures that sufficient strength is generated, provided the weld is properly sized. While penetration can be verified with ultrasonic inspection, fillet welds routinely receive only visual or magnetic particle inspection. Thus, no penetration beyond the root is required, nor is design credit given to deeper penetration in fillet welds if it happens to be present. Figure 3.17 illustrates this requirement.

The *out-of-position* capability of a given welding process refers to the ability to deposit weld metal in the vertical or overhead positions. It is generally more economical to position the work in the flat and horizontal positions. However, this is usually impossible for field erection, and may be impractical under other conditions. The ability to obtain *high travel speeds* is important for small welds. It may not be possible for a high-deposition welding process to be used at high travel speeds. The size of the droplet transferred, puddle fluidity, surface tension, and other factors combine to make some processes more capable of high travel speeds than others.

3.4.2 Process capabilities

After the joint is analyzed and specific requirements determined, these are compared to the capabilities of various processes. The process with capabilities most closely matching the requirements typically will be the best and most economical option.

Submerged arc welding and electroslag/electrogas welding have the greatest potential to deliver *high deposition rates*. Multiple-electrode applications of submerged arc extend this capability even further. For joints requiring high deposition rates, submerged arc and electroslag/electrogas welding are ideal processes to contribute to low-cost welding. When the specific conditions are not conducive to SAW but high deposition rates are still required, flux-cored arc welding may be used. The larger-diameter electrodes, which run at higher electrical currents, are preferred.

Deep penetration is offered by the submerged arc-welding process. While electroslag/electrogas also offers deep penetration, the joints on which the electroslag are used typically do not require this capability. Where open-arc processes are preferred, gas-shielded flux-cored arc welding may offer deep penetration.

Out-of-position capability is strongest for the flux-cored and shielded metal arc-welding processes. The slag coatings that are generated by these processes can be instrumental in retaining molten weld metal in the vertical and overhead positions. Submerged arc is not applicable for these joints.

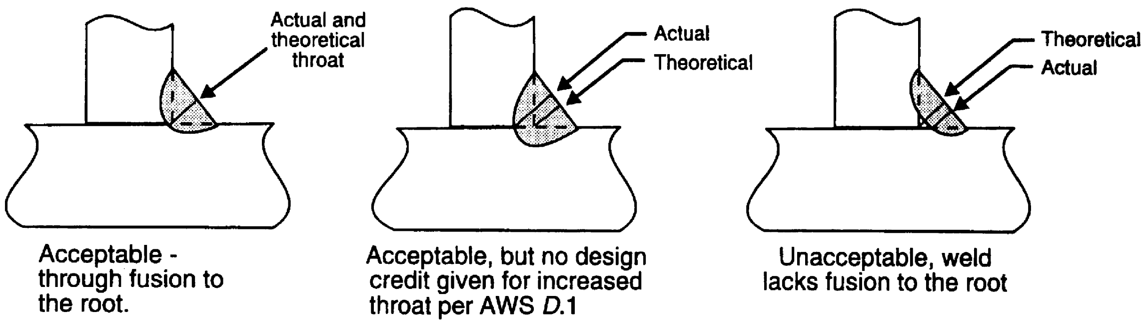


Figure 3.17 Fillet weld requirements. (*Courtesy of The Lincoln Electric Company.*)

The requirement for *high travel speed* capability for welding structural steel members is fairly limited. This typically consists of the travel speed associated with making a $\frac{1}{4}$-in fillet weld. All of the popular processes, with the exception of electroslag/electrogas, are capable of making $\frac{1}{4}$-in fillet welds under the proper conditions. Among the variables that need to be considered are electrode size and procedure variables. A common mistake of fabricators is to utilize a process and procedure capable of extremely high deposition rates but limited travel speeds. Oversized welds can result from the inability to achieve high travel speeds. A more economical approach would be to optimize the procedure according to the desired travel speed. This may result in a lower deposition rate but a lower overall cost because overwelding has been eliminated.

3.4.3 Special situations

Self-shielded flux-cored welding is ideal for *outdoor conditions*. Quality deposits may be obtained without the erection of special wind shields and protection from drafts. Shielded metal arc welding is also suitable for these conditions but is considerably slower.

The welding process of choice for field erectors for the last 25 years has been FCAW-ss. It has been the commonly used process for fabrication of steel structures throughout the United States. Its advantages are reviewed in order to provide an understanding of why it has been the preferred process. In addition, its limitations are outlined to highlight areas of potential concern.

The chief advantage of the FCAW-ss process is its ability to deposit quality weld metal under field conditions, which usually involve wind. The code specifically limits wind velocity in the vicinity of a weld to a maximum of 5 mi/h. In order to utilize gas-shielded processes under these conditions, it is necessary to erect windshields to preclude movement of the shielding gas with respect to the molten weld puddle. While tents and other housings can be created to minimize this problem, such activities can be costly and are often a fire hazard. In addition, adequate ventilation must be provided for the welder. The most efficient windshields may preclude adequate ventilation. Under conditions of severe shielding loss, weld porosity will be exhibited. At much lower levels of shielding loss, the mechanical properties (notch toughness and ductility) may be negatively affected, although there will be no obvious evidence that this is taking place.

A variety of other gas-related issues are also eliminated, including ensuring availability of gas, handling of high-pressure cylinders (always a safety concern), theft of cylinders, protection of gas-distribution hosing under field conditions, and the cost of shielding gas. Leaks in the delivery system obviously waste shielding gas, but a leak can also allow entry of air into the delivery system. Weld quality can be affected in the same way as shielding loss. Most field erectors have found it advantageous to utilize the self-shielded process and circumvent all such potential problems.

Some projects permit *multiple welding heads* to be simultaneously operated in the same general vicinity. When this is done, submerged arc is an ideal choice. Because of the lack of glare and arc flash, an operator can control multiple arcs that are nearly impossible to control in a situation where the arc intensity from one torch would make it difficult to carefully control another. A typical example would be the use of welding systems that simultaneously make fillet welds on opposing sides of stiffeners.

The easiest way to *control smoke and fumes* in the welding environment is to limit their initial generation. Here, submerged arc is ideal. Smoke exhaust guns are available for the flux-cored arc-welding processes. The most effective process for use with these smoke exhaust guns is FCAW-ss. Because the process is self-shielded, there is no concern about disruption of the gas shielding.

3.5 Welding Procedures

Within the welding industry, the term *welding procedure specification* (or WPS) is used to signify the combination of variables that are to be used to make a certain weld. The terms *welding procedure,* or simply *procedure,* may be used. At a minimum, the WPS consists of the following:

Process (SMAW, FCAW, etc.)

Electrode specification (AWS A5.1, A5.20, etc.)

Electrode classification (E7018, E71T-1, etc.)

Electrode diameter ($\frac{1}{8}$ in, $\frac{5}{32}$ in, etc.)

Electrical characteristics (ac, dc+, dc−)

Base metal specification (A36, A572 GR50, etc.)

Minimum preheat and interpass temperature

Welding current (amperage)/wire-feed speed

Arc voltage

Travel speed

Position of welding

Postweld heat treatment

Shielding gas type and flowrate

Joint design details

The welding procedure is somewhat analogous to a cook's recipe. It outlines the steps required to make a quality weld under specific conditions.

3.5.1 Effects of welding variables

The effects of the variables are somewhat dependent on the welding process being employed, but general trends apply to all the processes. It is important to distinguish the difference between constant current (CC) and constant voltage (CV) electrical welding systems. Shielded metal arc welding is always done with a CC system. Flux-cored welding and gas metal arc welding generally are performed with CV systems. Submerged arc may utilize either.

Amperage is a measure of the amount of current flowing through the electrode and the work. It is a primary variable in determining heat input. Generally, an increase in amperage means higher deposition rates, deeper penetration, and more admixture. The amperage flowing through an electric circuit is the same, regardless of where it is measured. It may be measured with a tong meter or with the use of an electric shunt. The role of amperage is best understood in the context of heat input and current density considerations. For CV welding, an increase in wire-feed speed will directly increase amperage. For SMAW on CC systems, the machine setting determines the basic amperage, although changes in the arc length (controlled by the welder) will further change amperage. Longer arc lengths reduce amperage.

Arc voltage is directly related to arc length. As the voltage increases, the arc length increases, as does the demand for arc shielding. For CV welding, the voltage is determined primarily by the machine setting, so the arc length is relatively fixed in CV welding. For SMAW on CC systems, the arc voltage is determined by the arc length, which is manipulated by the welder. As arc lengths are increased with SMAW, the arc voltage will increase and the amperage will decrease. Arc voltage also controls the width of the weld bead, with higher voltages generating wider beads. Arc voltage has a direct effect on the heat-input computation.

The voltage in a welding circuit is not constant, but is composed of a series of voltage drops. Consider the following example: Assume the power source delivers a total system voltage of 40 V. Between the power source and the welding head or gun, there is a voltage drop of perhaps 3 V associated with the input-cable resistance. From the point of attachment of the work lead to the power source work terminal, there is an additional voltage drop of, say, 7 V. Subtracting the 3 V and the 7 V from the original 40 V, this leaves 30 V for the arc.

This example illustrates how important it is to ensure that the voltages used for monitoring welding procedures properly recognize any losses in the welding circuit. The most accurate way to determine arc voltage is to measure the voltage drop between the contact tip and the workpiece. This may not be practical for semiautomatic welding, so voltage is typically read from a point on the wire feeder (where the gun and cable connection is made) to the workpiece. For SMAW, welding voltage is not usually monitored, since it is constantly changing and cannot be controlled except by the welder. Skilled workers hold short arc lengths to deliver the best weld quality.

Travel speed, measured in inches per minute, is the rate at which the electrode is moved relative to the joint. All other variables being equal, travel speed has an inverse effect on the size of the weld beads. As the travel speed increases, the weld size will decrease. Extremely low travel speeds may result in reduced penetration, as the arc impinges on a thick layer of molten metal and the weld puddle rolls ahead of the arc. Travel speed is a key variable used in computing heat input; reducing travel speed increases heat input.

Wire-feed speed is a measure of the rate at which the electrode is passed through the welding gun and delivered to the arc. Typically measured in inches per minute (in/min), the wire-feed speed is directly proportional to deposition rate and directly related to amperage. When all other welding conditions are maintained constant (for example, the same electrode type, diameter, electrode extension, arc voltage, and electrode extension), an increase in wire-feed speed will directly lead to an increase in amperage. For slower wire-feed speeds, the ratio of wire-feed speed to amperage is relatively constant and linear.

For higher levels of wire-feed speed, it is possible to increase the wire-feed speed at a disproportionately high rate compared to the increase in amperage. When these conditions exist, the deposition rate per amp increases but at the expense of penetration.

Wire-feed speed is the preferred method of maintaining welding procedures for constant-voltage wire-feed processes. The wire-feed speed can be independently adjusted, and measured directly, regardless of the other welding conditions. It is possible to utilize amperage as an alternative to wire-feed speed although the resultant amperage for a given wire-feed speed may vary, depending on the polarity, electrode diameter, electrode type, and electrode extension. Although equipment has been available for 20 years that monitors wire-feed speed, many codes such as AWS D1.1 continue to acknowledge amperage as the primary method for procedure documentation. D1.1 does permit the use of wire-feed speed control instead of amperage, providing a wire-feed speed–amperage relationship chart is available for comparison.

Electrode extension, also known as *stickout,* is the distance from the contact tip to the end of the electrode. It applies only to the wire-feed processes. As the electrode extension is increased in a constant-voltage system, the electrical resistance of the electrode increases, causing the electrode to be heated. This is known as *resistance heating* or I^2R heating. As the amount of heating increases, the arc energy required to melt the electrode decreases. Longer electrode extensions may be employed to gain higher deposition rates at a given amperage. When the electrode extension is increased without any change in wire-feed speed, the amperage will decrease. This results in less penetration and less admixture. With the increase in electric stickout, it is common to increase the machine voltage setting to compensate for the greater voltage drop across the electrode.

In constant-voltage systems, it is possible to simultaneously increase the electric stickout and wire-feed speed in a balanced manner so that the current remains constant. When this is done, higher deposition rates are attained. Other welding variables such as voltage and travel speed must be adjusted to maintain a stable arc and to ensure quality welding. The ESO variable should always be within the range recommended by the manufacturer.

Electrode diameter means larger electrodes can carry higher welding currents. For a fixed amperage, however, smaller electrodes result in higher deposition rates. This is because of the effect on current density discussed in the following.

Polarity is a definition of the direction of current flow. Positive polarity (reverse) is achieved when the electrode lead is connected to the positive terminal of the direct-current (dc) power supply. The work lead is connected to the negative terminal. Negative polarity (straight) occurs when the electrode is connected to the negative terminal and the work lead to the positive terminal. Alternating current (ac) is not a polarity, but a current type. With ac, the electrode is alternately positive and negative. Submerged arc is the only process that commonly uses either electrode positive or electrode negative polarity for the same type of electrode. AC may also be used. For a fixed wire-feed speed, a submerged arc electrode will require more amperage on positive polarity than on negative. For a fixed amperage, it is possible to utilize higher wire-feed speeds and deposition rates with negative polarity than with positive. AC exhibits a mix of both positive and negative polarity characteristics.

The magnetic field that surrounds any dc conductor can cause a phenomenon known as *arc blow,* where the arc is physically deflected by the field. The strength of the magnetic field is proportional to the square of the current value, so this is a more significant potential problem with higher currents. AC is less prone to art blow, and can sometimes be used to overcome this phenomenon.

Heat input is proportional to the welding amperage, times the arc voltage, divided by the travel speed. Higher heat inputs relate to larger weld cross-sectional areas and larger heat-affected zones, which may negatively affect mechanical properties in that region. Higher heat input generally results in slightly decreased yield and tensile strength in the weld metal, and generally lowers notch toughness because of the interaction of bead size and heat input.

Current density is determine by dividing the welding amperage by the cross-sectional area of the electrode. For solid electrodes, the current density is therefore proportional to I/d^2. For tubular electrodes where current is conducted by the sheath, the current density is related to the area of the metallic cross section. As the current density increases, there will be an increase in deposition rates, as well as penetration. The latter will increase the amount of admixture for a given joint. Notice that this may be accomplished by either the amperage or decreasing the electrode size. Because the electrode diameter is a squared function, a small decrease in diameter may have a significant effect on deposition rates and plate penetration.

Preheat and interpass temperature are used to control cracking tendencies, typically in the base materials. Regarding weld metal properties, for most carbon-manganese-silicon systems, a moderate interpass temperature promotes good notch toughness. Preheat and interpass temperatures greater than 550°F may negatively affect notch toughness. Therefore, careful control of preheat and interpass temperatures is critical.

3.5.2 Purpose of welding procedure specifications (WPSs)

The particular values for the variables discussed previously have a significant effect on weld soundness, mechanical properties, and productivity. It is therefore critical that those procedural values used in the actual fabrication and erection be appropriate for the specific requirements of the applicable code and job specifications. Welds that will be architecturally exposed, for example, should be made with procedures that minimize spatter, encourage exceptional surface finish, and have limited or no undercut. Welds that will be covered with fireproofing, in contrast, would naturally have less restrictive cosmetic requirements.

Many issues must be considered when selecting welding procedure values. While all welds must have fusion to ensure their strength, the required level of penetration is a function of the joint design in the weld type. All welds are required to deliver a certain yield and/or tensile strength, although the exact level required is a function of the connection design. Not all welds are required to deliver minimum specified levels of notch toughness. Acceptable levels of undercut and porosity are a function of the type of loading applied to the weld. Determination of the most efficient means by which these conditions can be met cannot be left to the welders, but should be determined by knowledgeable welding technicians and engineers who create written welding procedure specifications and communicate those requirements to welders by the means of these documents. The WPS is the primary tool that is used to communicate to the welder, supervisor, and inspector how a specific weld is to be made. The suitability of a weld made by a skilled welder in conformance with the requirements of a WPS can only be as good as the WPS itself. The proper selection of procedure variable values must be achieved in order to have a WPS appropriate for the application. This is the job of the welding expert who generates or writes the WPS. The welder is generally expected to be able to follow the WPS, although the welder may not know how or why each particular variable was selected. Welders are expected to ensure welding is performed in accordance with the WPS. Inspectors do not develop WPSs, but should ensure that they are available and are followed.

The *D1.1-98 Structural Welding Code—Steel* requires written welding procedures for all fabrication performed. The inspector is obligated to review the WPSs and to make certain that production welding parameters conform to the requirements of the code. These WPSs are required to be written, regardless of whether they are prequalified or qualified by test. Each fabricator or erector is responsible for the development of WPSs. Confusion on this issue apparently still exists since there continue to be reports of fabrication being performed in the absence of written welding procedure specifications. One prevalent misconception is that if the actual parameters under which welding will be performed meet all the conditions for "prequalified" status, written WPSs are not required. This is not true; according to the code, the requirement is clear.

The WPS is a communication tool, and it is the primary means of communication to all the parties involved regarding how the welding is to be performed. It must therefore be readily available to foremen, inspectors, and the welders.

The code is not prescriptive in its requirements regarding availability and distribution of WPSs. Some shop fabricators have issued each welder employed in their organization with a set of welding procedures that are typically retained in the welder's locker or tool box. Others have listed WPS parameters on shop drawings. Some company bulletin boards have listings of typical WPSs used in the organization. Some suggest that WPSs should be posted near the point where welding is being performed. Regardless of the method used, WPSs must be available to those authorized to use them.

It is in the contractor's best interest to ensure that efficient communication is maintained with all parties involved. Not only can quality be compromised when WPSs are not available, but productivity can suffer as well. Regarding quality, the limits of suitable operation of the particular welding process and electrode for the steel, joint design, and position of welding must be understood. It is obvious that the particular electrode employed must be operated on the proper polarity, proper shielding gases must be used, and amperage levels must be appropriate for the diameter of electrode and for the thickness of material on which welding is performed. Other issues are not necessarily so obviously apparent. The required preheat for a particular application is a function of the grade(s) of steel involved, the thickness(es) of material, and the type of electrode employed (whether low hydrogen or non-low hydrogen). The required preheat level can be communicated by means of the written WPS.

Lack of conformance with the parameters outlined in the WPS may result in the deposition of a weld that does not meet the quality requirements imposed by the code or the job specifications. When an unacceptable weld is made, the corrective measures to be taken may necessitate weld removal and replacement, an activity that routinely increases the cost of that particular weld tenfold. Avoiding these types of unnecessary activities by clear communication has obvious ramifications in terms of quality and economics.

There are other economic issues to be considered as well. In a most general way, the cost of welding is inversely proportional to the deposition rate. The deposition rate, in turn, is directly tied to the wire-feed speed of the semiautomatic welding processes. If it is acceptable, for example, to make a given weld with a wire-feed speed of 200 in/min, then a weld made at 160 in/min (which may meet all the quality requirements) would cost approximately 25% more than the weld made with the optimum procedure. Conformance with WPS values can help ensure that construction is performed at rates that are conducive to the required weld quality and are economical as well. Some wire feeders have the ability to preset welding parameters, coupled with the digital LED display or analog meters that indicate operational parameters, which can assist in maintaining and monitoring WPS parameters.

The code imposes minimum requirements for a given project. Additional requirements may be imposed by contract specifications. The same would hold true regarding WPS values. Compliance with the minimum requirements of the code may not be adequate under all circumstances. Additional requirements can be communicated through the WPS, such as recommendations imposed by the steel producer, electrode manufacturer, or others can and should be documented in the WPS.

3.5.3 Prequalified welding procedure specifications

The AWS D1.1 code provides for the use of prequalified WPSs. Prequalified WPSs are those that the AWS D1 Committee has determined to have a history of acceptable performance, and so does not subject them to the qualification testing imposed on all other welding procedures. The use of prequalified WPSs does not preclude their need to be in a written format. The use of prequalified WPSs still requires that the welders be appropriately qualified. All the workmanship provisions imposed in the fabrication section of the code apply to prequalified WPSs. The only code requirement exempted by prequalification is the nondestructive testing and mechanical testing required for qualification of welding procedures.

A host of restrictions and limitations imposed on prequalified welding procedures do not apply to welding procedures that are qualified by test. Prequalfied welding procedures must conform with all the prequalified requirements in the code. Failure to comply with a single prequalified condition eliminates the opportunity for the welding procedure to be prequalified. The use of a prequalified welding procedure does not exempt the engineer from exercising engineering judgment to determine the suitability of the particular procedure for the specific application.

In order for a WPS to be prequalified, the following conditions must be met:

- The welding process must be prequalified. Only SMAW, SAW, GMAW (except GMAW-s), and FCAW may be prequalified.
- The base metal/filler metal combination must be prequalified.
- The minimum preheat and interpass temperatures prescribed in D1.1-98 must be employed.
- Specific requirements for the various weld types must be maintained. Fillet welds must be in accordance with D1.1-98, Section 3.9, plug and slot welds in accordance with D1.1-98, and groove welds in accordance with D1.1-98, Sections 3.11 and 3.12, as applicable. For the groove welds, whether partial joint penetration or complete joint penetration, the required groove preparation dimensions are shown in the code.

Even if prequalified joint details are employed, the welding procedure must be qualified by test if other prequalified conditions are not met. For example, if a prequalified detail is used on an unlisted steel, the welding procedures must be qualified by test.

Prequalified status requires conformance to a variety of procedural parameters. These include maximum electrode diameters, maximum welding current, maximum root-pass thickness, maximum fill-pass thickness, maximum single-pass fillet weld sizes, and maximum single-pass weld layers.

In addition to all the preceding requirements, welding performed with a prequalified WPS must be in conformance with the other code provisions contained in the fabrication section of AWS *D1.1-98 Structural Welding Code.*

The code does not imply that a WPS that is prequalified will automatically achieve the quality conditions required by the code. It is the contractor's responsibility to ensure that the particular parameters selected within the requirements of the prequalified WPS are suitable for the specific application. An extreme example will serve as an illustration. Consider the following example of a hypothetical proposed WPS for making a $\frac{1}{4}$-in fillet weld on $\frac{3}{8}$-in A36 steel in the flat position. The weld type and steel are prequalified. SAW, a prequalified process, is selected. The filler metal selected is F7A2-EM12K, meeting the requirements of D1.1-98. No preheat is specified since it would not be required. The electrode diameter selected is $\frac{3}{32}$ in, less than the $\frac{1}{4}$-in maximum specified. The maximum single-pass fillet weld size in the flat position, according to D1.1-98 is unlimited, so the $\frac{1}{4}$-in fillet size can be prequalified. The current level selected for making this particular fillet weld is 800 A, less than the 1000-A maximum specified.

However, the amperage level imposed on the electrode diameter for the thickness of steel on which the weld is being made is inappropriate. It would not meet the requirements of the fabrication chapters which require that the size of electrode and amperage be suitable for the thickness of material being welded. This illustration demonstrates the fact that compliance with all prequalified conditions does not guarantee that the combination of selected variables will always generate an acceptable weld.

Most contractors will determine preliminary values for a prequalified WPS based upon their experience, recommendations from publications such as the AWS *Welding Handbooks,* from AWS *Welding Procedures Specifications* (AWS B2.1), or other sources. It is the responsibility of the contractor to verify the suitability of the suggested parameters prior to the application of the actual procedure on a project, although the verification test need not be subject to the full range of procedure qualification tests imposed by the code. Typical tests will be made to determine soundness of the weld deposit (for example, fusion, tie-in of weld beads, freedom from slag inclusions). The plate could be nondestructively tested or, as is more commonly done, cut, polished, and etched. The latter operations allow for examination of penetration patterns, bead shapes, and tie-in. Welds that are made with prequalified WPSs that meet the physical dimensional requirements (fillet weld size, maximum reinforcement levels, and surface profile requirements) and are sound (that is, adequate fusion, tie-in, and freedom from excessive slag inclusions and porosity) should meet the strength and ductility requirements imposed by the code for welding procedures qualified by test. Weld soundness, however, cannot be automatically assumed just because the WPS is prequalified.

3.5.4 Guidelines for preparing prequalified WPSs

When developing prequalified WPSs, the starting point is a set of welding parameters appropriate for the general application being considered. Parameters for overhead welding will naturally vary from those required for down-hand welding. The thickness of material involved will dictate electrode sizes and corresponding current levels. The specific filler metals selected will reflect the strength requirements of the connection. Many other issues must be considered. Depending on the level of familiarity and comfort the contractor has with the particular values selected, welding a mock-up may be appropriate. Once the parameters that are desired for use in production are established, it is essential to check each of the applicable parameters for compliance with the D1.1-98 code.

To assist in this effort, Annex H has been provided in the D1.1-98 code. This contains a checklist that identifies prequalified requirements. If any single parameter deviates from these requirements, the contractor is left with two options: (1) the preliminary procedure can be adjusted to conform with the prequalified constraints or (2) the WPS can be qualified by test. If the preliminary procedure is adjusted, it may be appropriate to reexamine its viability by another mock-up.

The next step is to document, in writing, the prequalified WPS values. A sample form is included in Annex E of the code. The fabricator may utilize any convenient format. Also contained in Annex E are a series of examples of completed WPSs that may be used as a pattern.

3.5.5 Qualifying welding procedures by test

Conducting qualification tests. There are two primary reasons why welding procedures may be qualified by test. First, it may be a contractual requirement. Secondly, one or more of the specific conditions to be used in the production may deviate from the prequalified requirements. In either case, a test weld must be made prior to the establishment of the final WPS. The first step in qualifying a welding procedure by test is to establish the procedure that is desired to be qualified. The same sources cited for the prequalified WPS starting points could be used for WPSs qualified by test. These will typically be the parameters used for fabrication of the test plate, although this is not always the case, as will be discussed later. In the simplest case, the exact conditions that will be encountered in production will be replicated in the procedure qualification test. This would include the welding process, filler metal, grade of steel, joint details, thickness of material, preheat values, minimum interpass temperature level, and the various welding parameters of amperage, voltage, and travel speed. The initial parameters used to make the procedure qualification test plate beg for a name to define them, although there is no standard industry term. It has been suggested that "TWPS" be used where the "T" could alternatively be used for temporary, test, or trial. In any case, it would define the parameters to be used for making the test plate since the validity of the particular parameters cannot be verified until successfully passing the required test. The parameters for the test weld are recorded on a procedure qualification record (PQR). The actual values used should be recorded on this document. The target voltage, for example, may be 30 V but, in actual fact, only 29 V were used for making the test plate. The 29 V would be recorded.

After the test plate has been welded, it is allowed to cool and the plate is subjected to the visual and nondestructive testing as prescribed by the code. The specific tests required are a function of the type of weld being made and the particular welding consumables. The types of qualification tests are described in D1.1-98, paragraph 4.4.

In order to be acceptable, the test plates must first pass visual inspection followed by nondestructive testing (NDT). At the contractor's option, either RT or UT can be used for NDT. The mechanical tests required involve bend tests (for soundness) macroetch tests (for soundness), and reduced section tensile tests (for strength). For qualification of procedures on steels with significantly different mechanical properties, a longitudinal bend specimen is possible. All weld metal tensile tests are required for unlisted filler metals. The nature of the bend specimens, whether side, face, or root, is a function of the thickness of the steel involved. The number and type of tests required are defined in D1.1-98, Table 4.2, for complete joint penetration groove welds; D1.1-98, Table 4.3, for partial joint penetration groove welds; and D1.1-98, Table 4.4, for fillet welds.

Once the number of tests has been determined, the test plate is sectioned and the specimens machined for testing. The results of the tests are recorded on the PQR. According to D1.1-98, if the test results meet all the prescribed requirements, the testing is successful and welding procedures can be established based upon the successful PQR. If the test results are unsuccessful, the PQR cannot be used to establish the WPS. If any one specimen of those tested fails to meet the test requirements, two retests of that particular type of test may be performed with specimens extracted from the same test plate. If both of the supplemental specimens meet the requirements, the D1.1-98 allows the tests to be deemed successful. If the test plate is over 1½ in thick, failure of a specimen necessitates retesting of all the specimens at the same time from two additional locations in the test material.

It is wise to retain the PQRs from unsuccessful tests as they may be valuable in the future when another similar welding procedure is contemplated for testing.

The acceptance criteria for the various tests are prescribed in the code. The reduced section tensile tests are required to exceed the minimum specified tensile strength of the steel being joined. Specific limits on the size, location, distribution, and type of indication on bend specimens are prescribed in D1.1-98, paragraph 4.8.3.3.

Writing WPSs from successful PQRs. When a PQR records the successful completion of the required tests, welding procedures may be written from that PQR. At a minimum, the values used for the test weld will constitute a valid WPS. The values recorded on the PQR are simply transcribed to a separate form, now known as a WPS rather than a PQR.

It is possible to write more than one WPS from a successful PQR. Welding procedures that are sufficiently similar to those tested can be supported by the same PQR. Significant deviations from those conditions, however, necessitate additional qualification testing. Changes that are considered significant enough to warrant additional testing are considered essential variables, and these are listed in D1.1-98, Tables 4.5, 4.6, and 4.7. For example, consider an SMAW welding procedure that is qualified by test using an E8018-C3 electrode. From that test, it would be possible to write a WPS that utilizes E7018 (since this is a decrease in electrode strength) but it would not be permissible to write a WPS that utilizes E9018-G electrode (because Table 4.5 lists an increase in filler metal classification strength as an essential variable). It is important to carefully review the essential variables in order to determine whether a previously conducted test may be used to substantiate the new procedure being contemplated.

D1.1-98, Table 4.1, defines the range of weld types and positions qualified by various tests. This table is best used, not as an after-the-fact evaluation of the extent of applicability of the test already conducted, but rather for planning qualification tests. For example, a test plate conducted in the 2G position qualifies the WPS for use in either the 1G or 2G position. Even though the first anticipated use of the WPS may be for the 1G position, it may be advisable to qualify in the 2G position so that additional usage can be obtained from this test plate.

In a similar way, D1.1-98, Table 4.7, defines what changes can be made in the base metals used in production versus qualification testing. An alternative steel may be selected for the qualification testing simply because it affords additional flexibility for future applications.

If WPS qualification is performed on a nonprequalified joint geometry, and acceptable test results are obtained, WPSs may be written from that PQR utilizing any of the prequalified joint geometries (D1.1-98, Table 4.5, item 32).

3.5.6 Approval of WPSs

After a WPS is developed by the fabricator or erector, it is required to be reviewed in accordance to D1.1 requirements. For prequalified WPSs, the inspector is required to review the WPSs to ensure that they meet all the prequalified requirements. For WPSs that are qualified by test, the AWS D1.1-98 code requires these to be submitted to the engineer for review.

The apparent logic behind the differences in approval procedures is that while prequalified WPSs are based upon well-established time-proven, and documented welding practices, WPSs that have been qualified by test are not automatically subject to such restrictions. Even though the required qualification tests have demonstrated the adequacy of the particular procedure under test conditions, further scrutiny by the engineer is justified to ensure that it is applicable for the particular situation that will be encountered in production.

In practice, it is common for the engineer to delegate the approval activity of all WPSs to the inspector. There is a practical justification for such activity: the engineer may have a more limited understanding of welding engineering, and the inspector may be more qualified for this function. While this practice may be acceptable for typical projects that utilize common materials, more scrutiny is justified for unusual applications that utilize materials in ways that deviate significantly from normal practice. In such situations, it is advisable for the engineer to retain the services of a welding expert to evaluate the suitability of the WPSs for the specific application.

3.6 Weld Size Determination

3.6.1 Strength of welded connections

A welded connection can be designed and fabricated to have a strength that matches or exceeds that of the steel it joins. This is known as a *full-strength connection* and can be considered 100% efficient. Welded connections can be designed so that if loaded to destruction, failure would occur in the base material. Poor weld quality, however, may adversely affect weld strength. Porosity, slag inclusions, and lack of fusion may decrease the capacity of a complete joint penetration (CJP) groove weld.

A connection that duplicates the base metal capacity is not always necessary and when unwarranted, its specification unnecessarily increases fabrication costs. In the absence of design information, it is possible to design welds that have strengths equivalent to the base material capacity. Assuming the base plate has been properly selected, a weld sized around the base plate will be adequate as well. This, however, is a very costly approach. Economical welded structures cannot be designed on this basis. Unfortunately, the overuse of the CJP detail and the requirement of "matching filler metal" serves as evidence that this is often the case.

3.6.2 Variables affecting welded connection strength

The strength of a welded connection is dependent on the weld metal strength and the area of weld that resists the load. Weld metal strength is a measure of the capacity of the deposited weld metal itself, measured in units such as ksi (kips per square inch). The connection strength reflects the combination of weld metal strength and cross-sectional area, and would be expressed as a unit of force, such as kips. If the product of area times the weld metal strength exceeds the load applied, the weld should not fail in static service.

The area of weld metal that resists fracture is the product of the theoretical throat multiplied by the length. The *theoretical weld throat* is defined as the minimum distance from the root of the weld to its theoretical face. For a CJP groove weld, the theoretical throat is assumed to be equal to the thickness of the plate it joins. Theoretical throat dimensions of several types of welds are shown in Fig. 3.18.

For fillet welds or partial joint penetration groove welds, using filler metal with strength levels equal to or less than the base metal, the theoretical failure plane is through the weld throat. When the same weld is made using filler metal with a strength level greater than that of the base metal, the failure plane may shift into the fusion boundary or heat-affected zone. From a design perspective, this is an undesirable condition and may lead to performance problems.

Complete joint penetration groove welds that utilize weld metal with strength levels exactly equal to the base metal will theoretically fail in either the weld or the base metal. Since the weld metal is generally slightly higher in strength than the base metal, the theoretical failure plane for transversely loaded connections is assumed to be in the base metal.

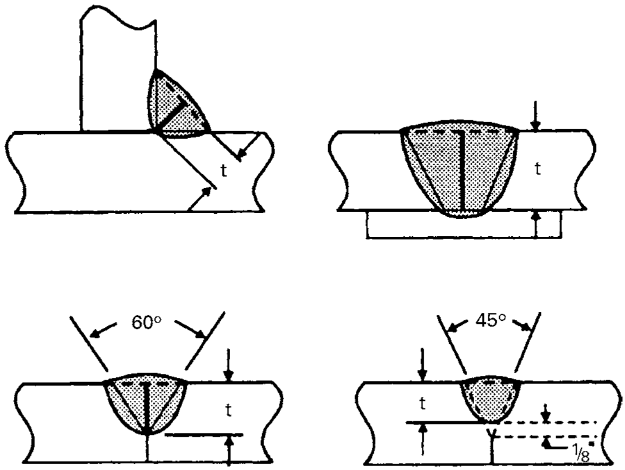


Figure 3.18 Theoretical throats. (*Courtesy of The Lincoln Electric Company.*)

In review, connection strength is governed by three variables: weld metal strength, weld length, and weld throat. The highest value of weld metal strength to be used for these calculations is a value comparable to the base metal. The weld length is often fixed, due to the geometry of the parts being joined, leaving one variable to be determined, namely, the throat dimension.

3.6.3 Determining throat size for tension or shear loads

For tension or shear loads, the required capacity the weld must deliver is simply the force divided by the length of the weld. The result, in units of force per length (such as kips per inch) can be divided by the weld metal capacity, in units of force per area (such as kips per square inch). The final result would be the required throat, in inches. Weld metal allowables which incorporate factors of safety can be used instead of the actual weld metal capacity. This directly generates the required throat size.

To determine the weld size, it is necessary to consider what type of weld is to be used. Assume the preceding calculation determined the need for a 1-in throat size. If a single fillet weld is to be used, a throat of 1.0 in would necessitate a leg size of 1.4 in, shown in Fig. 3.19. For double-sided fillets, two 0.7-in leg size fillets could be used. If a single PJP groove weld is used, the effective throat would have to be 1.0 in. The actual depth of preparation of the production joint would be 1.0 in or greater, depending on the welding procedure and included angle used. A double PJP groove weld would require two effective throats of

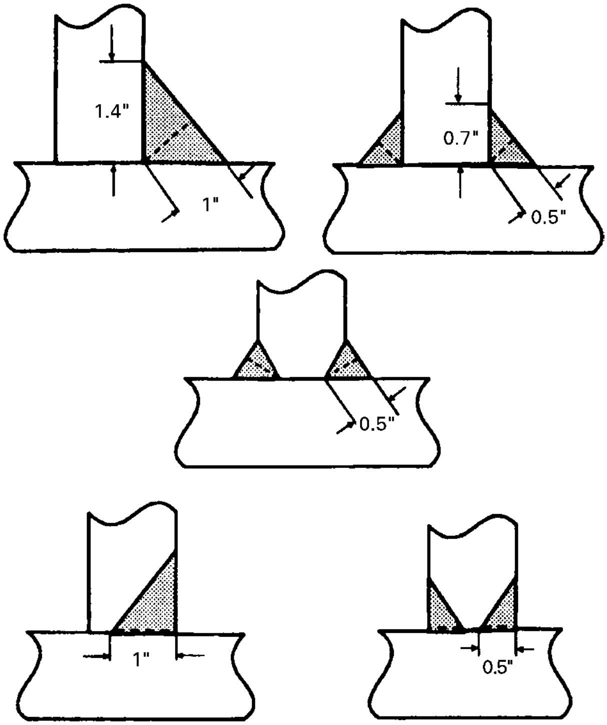


Figure 3.19 Weld combinations with equal throat dimensions. (*Courtesy of The Lincoln Electric Company.*)

0.5 in each. A final option would be a combination of partial joint penetration groove welds and external fillet welds. As shown in Fig. 3.11, a 60° included angle was utilized for the PJP groove weld and an unequal leg fillet weld applied externally. This acts to shift the effective throat from the normal 45° angle location to a 30° throat.

If the plates being joined are 1.0 in thick, a CJP groove weld is the only type of groove weld that will effectively transfer the stress, since the throat on a CJP weld is equal to the plate thickness. PJP groove welds would be incapable of developing adequate throat dimensions for this application, although the use of a combination PJP fillet weld would be a possibility.

3.6.4 Determining throat size for compressive loads

When joints are only subject to compression, the unwelded portion of the joint may be milled-to-bear, reducing the required weld throat. Typical of these types of connections are column splices where partial joint penetration (PJP) groove welds frequently are used for static structures. In dynamic structures subject to many compression cycles, CJP groove welds often are required for fatigue reasons.

In theory, compression joints require no welds, providing the base metals will bear on another bearing surface. Some horizontal shearing forces may be present and the use of a weld with a throat equal to 50% of the base metal thickness is common.

3.6.5 Practical approach to determine weld size for bending or torsional loads

The following is a simple method to determine the correct amount of welding required for adequate strength for a bending or torsional load. This is a method in which the weld is treated as a line, having no area, but a definite length and outline. This method has the following advantages:

1. It is not necessary to consider throat areas because only a line is considered.
2. Properties of weld are easily found from a table without knowing weld leg size.
3. Forces are considered on a unit length of weld instead of stresses, thus eliminating the knotty problem of combining stresses.
4. Actual test values of welds are given as force per unit length of weld instead of unit stress on throat of weld.

3.6.6 Treat weld as a line

Visualize the welded connection as a single line, having the same outline as the connection, but no cross-sectional area. Notice (Fig. 3.20)

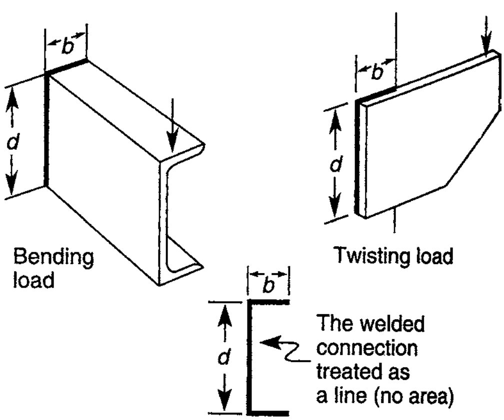


Figure 3.20 Treating weld as line.

that the area of the welded connection now becomes just the length of the weld.

Instead of trying to determine the stress on the weld (this cannot be done unless the weld size is known), the problem becomes a much simpler one of determining the force on the weld.

3.6.7 Use standard formulas to find force on weld

When the weld is treated as a line by inserting the property of the welded connection into the standard design formula used for that particular type of load (see Table 3.1), the force on the weld may be found in terms of pounds per lineal inch of weld.

For example, for bending:

Standard design formula (bending stress)	Same formula used for weld (treating weld as a line)
$\sigma = \dfrac{M}{S}$	$f = \dfrac{M}{S_w}$

Normally the use of these standard design formulas results in a *unit stress,* in pounds per square inch; however, when the weld is treated as a line, these formulas result in a *force* on the weld, in pounds per lineal inch.

For secondary welds, the weld is not treated as a line, but standard design formulas are used to find the force on the weld, in pounds per lineal inch.

In problems involving bending or twisting loads, Table 3.2 is used. It contains the section modulus, S_w, and polar moment of inertia, J_w, of some 13 typical welded connections with the weld treated as a line.

For any given connection, two dimensions are needed, width b and depth d.

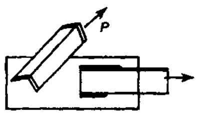
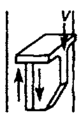
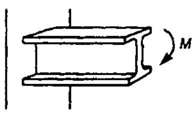
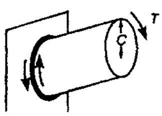
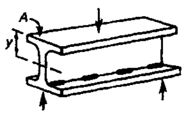
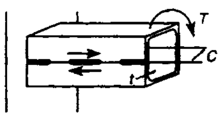
Section modulus S_w is used for welds subjected to bending loads, and polar moment of inertia J_w for twisting loads.

Section moduli S_w from these formulas are for maximum force at the top as well as the bottom portions of the welded connections. For the unsymmetrical connections shown in the table, maximum bending force is at the bottom.

If there is more than one force applied to the weld, these are found and combined together. All forces which are combined (vectorially added) must occur at the same position in the welded joint.

Calculating weld size for longitudinal welds. Longitudinal welds constitute the majority of the welding performed in many types of construction and hence justify special emphasis. These include the

TABLE 3.1 Treating a Weld as a Line

Type of loading		Standard design formula	Treating the weld as a line
		Stress lb/in^2	Force, lb/in
Primary welds transmit entire load at this point			
	Tension or compression	$\sigma = \frac{P}{A}$	$f = \frac{P}{A_w}$
	Vertical shear	$\sigma = \frac{V}{A}$	$f = \frac{V}{A_w}$
	Bending	$\sigma = \frac{M}{S}$	$f = \frac{M}{S_w}$
	Twisting	$\sigma = \frac{TC}{J}$	$f = \frac{TC}{J_w}$
Secondary welds hold section together—low stress			
	Horizontal shear	$\tau = \frac{VA_y}{It}$	$f = \frac{VA_y}{In}$
	Torsion horizontal shear*	$\tau = \frac{T}{2At}$	$f = \frac{T}{2A}$

A = area contained within median line.
* Applies to closed tubular section only.

b = width of connection, in
d = depth of connection, in
A = area of flange material held by welds in horizontal shear, in^2
y = distance between center of gravity of flange material and N.A. of whole section, in
I = moment of inertia of whole section, in.^{4}
C = distance of outer fiber, in
t = thickness of plate, in
J = polar moment of inertia of section, in.^{4}
P = tensile or compressive load, lb
V = vertical shear load, lb
M = bending moment, in·lb
T = twisting moment, in·lb.
A_w = length of weld, in
S_w = section modulus of weld, in^2
J_w = polar moment of inertia of weld, in^3
N_x = distance from x axis to face
N_y = distance from y axis to face
S = stress in standard design formula, lb/in^2
f = force in standard design formula when weld is treated as a line, lb/in
n = number of welds

TABLE 3.2 Treating a Weld as a Line

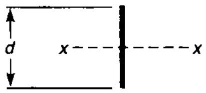
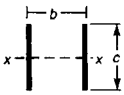
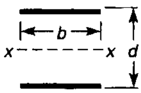
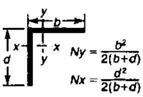
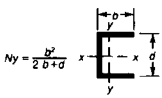
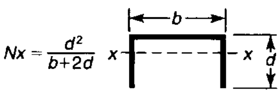
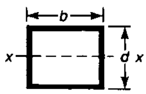
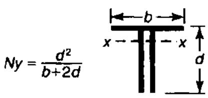
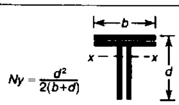
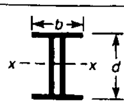
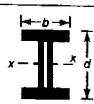
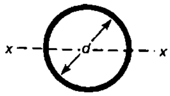
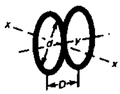
Outline of welded joint b = width d = depth	Bending (about horizontal axis $x - x$)	Twisting
	$S_w = \frac{d^2}{6}$ in^2	$J_w = \frac{d^3}{12}$ in^3
	$S_w = \frac{d^2}{3}$	$J_w = \frac{d(3b^2 + d^2)}{6}$
	$S_w = bd$	$J_w = \frac{b^3 + 3bd^2}{6}$
	$S_w = \frac{4bd + d^2}{6} = \frac{d^2(4b + d)}{6(2b + d)}$ top bottom	$J_w = \frac{(b + d)^4 - 6b^2d^2}{12(b + d)}$
	$S_w = bd + \frac{d^2}{6}$	$J_w = \frac{(2b + d)^3}{12} - \frac{b^2(b + d)^2}{2b + d}$
	$S_w = \frac{2bd + d^2}{3} = \frac{d^2(2b + d)}{3(b + d)}$ top bottom	$J_w = \frac{(b + 2d)^3}{12} - \frac{d^2(b + d)^2}{b + 2d}$
	$S_w = bd + \frac{d^2}{3}$	$J_w = \frac{(b + d)^3}{6}$
	$S_w = \frac{2bd + d^2}{3} = \frac{d^2(2b + d)}{3(b + d)}$ top bottom	$J_w = \frac{(b + 2d)^3}{12} - \frac{d^2(b + d)^2}{b + 2d}$
	$S_w = \frac{4bd + d^2}{3} = \frac{4bd^2 + d^3}{6b + 3d}$ top bottom	$J_w = \frac{d^3(4b + d)}{6(b + d)} + \frac{b^3}{6}$
	$S_w = bd + \frac{d^2}{3}$	$J_w = \frac{b^3 + 3bd^2 + d^3}{6}$
	$S_w = 2bd + \frac{d^2}{3}$	$J_w = \frac{2b^3 + 6bd^2 + d^3}{6}$

TABLE 3.2 Treating a Weld as a Line (*Continued*)

	$S_w = \frac{\pi d^2}{4}$	$J_w = \frac{\pi d^3}{4}$
	$I_w = \frac{\pi d}{2}\left(D^2 + \frac{d^2}{2}\right)$ $S_w = \frac{I_w}{c}$ where $c = \frac{\sqrt{D^2 + d^2}}{2}$	

web-to-flange welds on I-shaped girders, and the welds on the comers of the box girders. These welds primarily transmit horizontal shear forces resulting from the change in moment along the member. To determine the force between the members being joined, the following equation may be used:

$$f = \frac{Vay}{In}$$

where f = force on weld
V = total shear on section at a given position along the beam
a = area of flange connected by the weld
y = distance from the neutral axis of the whole section to the center of gravity of the flange
I = moment of inertia of the whole section
n = number of welds joining the flange to webs per joint

The resulting force is then divided by the allowable stress in the weld metal and the weld throat is attained. This particular procedure is emphasized because the resultant value for the weld throat is nearly always less than the minimum allowable weld size. The minimum size then becomes the controlling factor.

3.6.8 Filler metal strength requirements

Filler metal strength may be classified as "matching," "undermatching," or "overmatching." *Matching filler metal* has the same or slightly higher minimum specified yield and tensile strength compared to the minimum specified properties of the base material. Emphasis is placed on minimum specified properties because actual properties are routinely higher. Matching filler metal for A572 GR50 would be E70 material, where the minimum specified filler metal/base metal properties for yield are 60/50 ksi and for tensile are 70/65 ksi. Even though the filler metal has slightly higher properties than the base metal, this is considered to be a matching combination.

Many see the filler metal recommendations provided in codes that reference "matching" combinations for various grades of steel and assume that is the only option available. While this will never generate a nonconservative answer, it may eliminate better options. Matching filler metal tables are designed to give recommendations for the one unique situation where matching is required (for example, CJPs in tension). Other alternatives should be considered, particularly when the residual stresses on the welded connection can be reduced in crack-sensitive or distortion-prone configurations.

Matching filler metal is required for CJP groove welds loaded in tension. In order to achieve a full-strength welded connection, the filler metal must have a strength that is at least equal to that of the material it joins.

Undermatching weld metal may be used for all weld types and loading types except one: complete joint penetration groove welds loaded in tension. For all other joints and other loading types, some degree of undermatching is permitted. For example, CJPs in compression may be made with weld metal that has a strength of up to 10 ksi less than matching. CJPs in shear or loading parallel to the longitudinal axis may be made with undermatching filler material. All PJPs, fillet welds, and plug or slot welds may be made with undermatching weld metal. Design of the weld sizes, however, must incorporate the lower filler metal strength in order to ensure the welded connection has the proper capacity.

Undermatching may be used to reduce the concentration of stresses in the base material. Lower-strength filler material generally will be more ductile than higher strength weld metal. In Fig. 3.21, the first weld was made with matching filler material. The second design utilizes undermatching weld metal. To obtain the same capacity in the

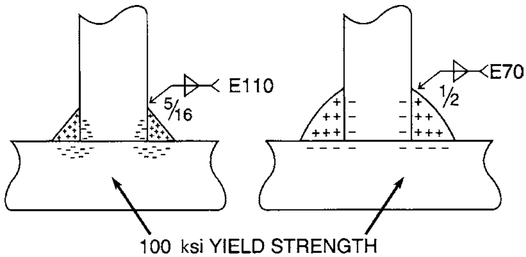


Figure 3.21 Matching and undermatching filler metal. (*Courtesy of The Lincoln Electric Company.*)

second joint, a larger fillet weld has been specified. Since the residual stresses are assumed to be of the order of the yield point of the weaker material in the joint, the first example would have residual stresses in the weld metal and the base metal of approximately 100 ksi. In the second example, the residual stresses in the base material would be approximately 60 ksi, since the filler metal has the lower yield point. These lower residual stresses will reduce cracking tendencies.

In situations where the weld size is controlled by the minimum permitted size, undermatching is a particularly desirable option. If a ¼-in fillet weld is required because of the minimum fillet weld size, it may be made of undermatching weld material without increasing the weld size due to the undermatching requirement.

Overmatching weld metal should be discouraged. It offers no advantages, and will increase residual stresses and distortion. Higher yield-strength weld metal generally is less ductile and more crack sensitive. Exceptions to this guideline are the filler materials used to join A588 weathering steel. In the process of adding alloys for atmospheric corrosion resistance, most filler metals for weathering steel will deposit 80-ksi tensile strength weld metal. Compared to the 70-ksi tensile strength weathering steel, this is an overmatch. The combination, however, performs well and because of the limited alternatives, this slight overmatch is permitted.

Caution must be exercised when overmatching filler metal is deliberately used. The strength of fillet and PJP groove welds is controlled by the throat dimension, weld length, and capacity of the weld metal. In theory, overmatching filler metal would enable smaller weld sizes to be employed and yet create a weld of equal strength. However, the strength of a connection is dependent not only on the weld strength but also on the strength of the fusion zone. As weld sizes are reduced, the fusion zone is similarly decreased in size. The capacity of the base metal is not affected by the selection of filler metal, so it remains unchanged. The reduction in weld size may result in an overstressing of the base metal.

Consider three tee joints containing PJP groove welds and illustrated in Fig. 3.22. A load is applied parallel to the weld, that is, the weld is subject to shear. The allowable stress on the groove weld is 30% of the nominal strength of the weld metal, that is, the "E" number (for example, E60, E70, etc.). Allowable stress on the base metal shall not exceed 40% of the yield strength of the base metal. The first combination employs a very close match of weld metal to base metal, namely, A572 GR50 welded with E70 electrode. The second example examines the same steel welded with undermatching E60 electrode, and the final illustration shows an example of overmatching with E80 electrode.

The weld capacity, in kips per inch, has been determined by multiplying the weld throat by the allowable stress. In the undermatching

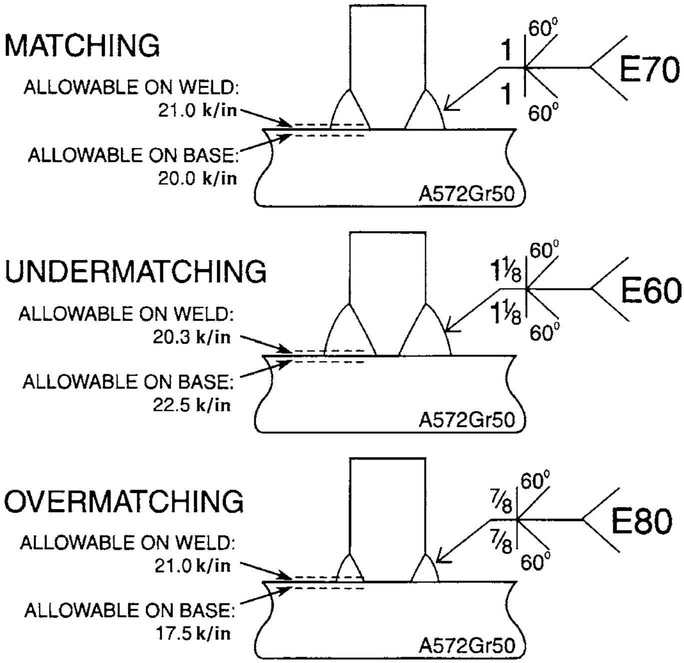


Figure 3.22 Effect of filler metal strength level. (*Courtesy of The Lincoln Electric Company.*)

case, notice that the weld controls. If the weld is properly designed, the base metal will not be overstressed. With matching weld metal, the loading on both the weld and the base metal is essentially the same. But, in the case of the overmatching combination, the weld has 20% more capacity than the base metal. If a designer overlooked loading on the base metal, the connection could easily be overlooked.

It should be noted, however, that all filler metal combinations will overmatch A36. Particular caution should be taken when sizing PJP groove welds on this steel to ensure that the base metal allowables are not exceeded.

Another area of potential problem is the slightly overmatched alloy filler metals used on A588. For PJP groove welds subject to shear loading, weld sizes that are determined based upon E80 filler metal will result in an overstressing of the base metal. However, the problem is eliminated when design calculations are made based on E70 filler metal. This is the recommended approach since some acceptable filler metals for weathering applications are classified E70. The base metal will not be overstressed, and the fabricator will have the flexibility of employing either E70 or E80 filler metal.

3.7 Welding Cost Analysis

Welding is a labor-intensive technology. Electricity, equipment depreciation, electrodes, gases, and fluxes constitute a very small portion of the total welding cost. Therefore, the prime focus of cost control will be on reducing the amount of time required to make a weld.

The following example is given to illustrate the relative costs of material and labor, as well as to assess the effects of proper process selection. The example to be considered is the groove weld of beam flange-to-column connections. Since this is a multiple-pass weld, the most appropriate analysis method is to consider the welding cost per weight of weld metal deposited, such as dollars per pound. Other analysis methods include cost per piece, ideal for manufacturers associated with the production of identical parts on a repetitive basis, and cost per length, appropriate for single-pass welds of substantial length. The two welding processes to be considered are shielded metal arc welding and flux-cored arc welding. Either would generate high-quality welds when properly used.

To calculate the cost per weight of weld metal deposited, an equation taking the following format is used:

$$\text{Cost per weight} = \frac{\text{electrode cost}}{\text{efficiency}} + \frac{\text{labor + overhead rate}}{\text{(deposition rate)(operating factor)}}$$

The cost of the electrode is simply the purchase cost of the welding consumable used. Not all of this filler metal is converted directly to deposited weld metal. There are losses associated with slag, spatter, and in the case of SMAW, the stub loss (the end portion of the electrode that is discarded). To account for these differences, an efficiency factor is applied. The following efficiency factors are typically used for the various welding processes:

Process	Efficiency, %
SMAW	60
FCAW	80
GMAW	90 (CO_2 shielding)
	98 (mixed gas)
SAW	100 (flux not included)

The cost to deposit the weld metal is determined by dividing the applicable labor and overhead rate by the deposition rate, that is, the amount of weld metal deposited in a theoretical, continuous 1 h of production. This cannot be maintained under actual conditions since welding will be interrupted by many factors, including slag removal, replacement of electrode, repositioning of the work or the welder with respect to the work, etc. To account for this time, an "operating factor" is used which is defined as the *arc-on* time divided by the total time associated with welding activities. For SMAW, replacement of electrodes takes place approximately every minute because of the finite length of the electrodes used. The following operating factors are typically used for the various processes and method of application:

Method	Operating factor, %
Manual SMAW	30
Semiautomatic	40
Mechanized	50

Operating factors for any given process can vary widely, depending on what a welder is required to do. In shop situations, a welder may receive tacked assemblies and be required only to weld and clean them. For field erection, the welder may "hang iron," fit, tack, bolt, clean the joint, reposition scaffolding, and perform other activities in addition to welding. Obviously, operating factors will be significantly reduced under these conditions.

The following examples are the actual procedures used by a field erector. The labor and overhead costs do not necessarily represent actual practice. The operating factors are unrealistically high for a field erection site, but have been used to enable comparison of the relative cost of filler metals versus the labor required to deposit the weld metal, as well as the difference in cost for different processes. Once the cost per deposited pound is known, it is relatively simple to determine the quantity of weld metal required for a given project, and multiply it by the cost per weight to determine the cost of welding on the project.

Process	SMAW	FCAW
Electrode classification	E7018	E70TG-K2
Electrode diameter, in	$^3/_{16}$	$^7/_{64}$
Amperage	225	430
Voltage	NA	27
Electrode efficiency, %	60	80
Electrode cost, $/lb	1.23	2.27
Operating factor, %	30	40
Deposition rate, lb/h	5.5	14.5
Labor and overhead rate, $/h	50	50

For SMAW:

$$\text{Cost per weight} = \frac{\$1.23}{60\%} + \frac{\$50.00}{(5.5)(30\%)} = \$2.05 + \$30.30 = \$32.35/\text{lb}$$

For FCAW:

$$\text{Cost per weight} = \frac{\$2.27}{80\%} + \frac{\$50.00}{(14.5)(40\%)} = \$11.46 + \$8.62 = \$11.46/\text{lb}$$

In the SMAW example, the electrode cost is approximately 6% of the total cost. For the FCAW example, primarily due to a decrease in the labor content, the electrode cost is 25% of the total. By using FCAW, the total cost of welding was decreased approximately 65%. While the FCAW electrode costs 85% more than the SMAW electrode, the higher electrode efficiency reduces the increase in electrode cost to only 39%. The first priority that must be maintained when selecting welding processes and procedures is the achievement of the required weld quality. For different welding methods which deliver the required quality, it is generally advantageous to utilize the method that results in higher deposition rates and higher operating factors. This will result in reduced welding time with a corresponding decrease in the total building erection cycle, which will generally translate to a direct savings for the final owner, not only lowering the cost of direct labor, but also reducing construction loan costs.

3.8 Techniques to Limit Distortion

3.8.1 Why distortion occurs

Distortion occurs due to the nonuniform expansion and contraction of weld metal and adjacent base material during the heating and cooling cycles of the welding process. At elevated temperatures, hot, expanded weld and base metal occupies more physical space than it will at room temperatures. As the metal contracts, it induces strains that result in stresses being applied to the surrounding base materials. When the surrounding materials are free to move, distortion results. If they are not free to move, as in the case of heavily restrained materials, these strains can induce cracking stresses. In many ways, distortion and cracking are related. It should be emphasized that not only the weld metal, but also the surrounding base material, is involved in this contraction process. For this reason, welding processes and procedures that introduce high amounts of energy into the surrounding base material will cause more distortion. Stresses resulting from material shrinkage are inevitable in welding. Distortion, however, can be minimized, compensated for, and predicted. Through efficient planning, design, and fabrication practices, distortion-related problems can be effectively minimized.

3.8.2 Control of distortion

Design concepts to minimize distortion. The engineer who is aware of the effects of distortion can design measures into the welded assemblies that will minimize the amount of distortion. These concepts include the following:

Minimize the amount of weld metal: Any reduction in the amount of weld metal will result in a decrease in the amount of distortion:

- Use the smallest acceptable weld size
- Use intermittent welds where acceptable
- Utilize double-sided joints versus single-sided joints where applicable
- Use groove details that require the minimum volume of weld per metal per length

Fabrication practices that minimize distortion. Fabricators can use techniques that will minimize distortion. These include the following:

Use as few weld passes as possible: Fewer passes are desirable inasmuch as they limit the number of heating and cooling cycles to which the joint will be subjected. The shrinkage stresses of each pass tend to accumulate, increasing the amount of distortion when many passes are used. Note that this is in direct contrast with the criterion of maximizing notch toughness.

Avoid overwelding: Overwelding results in more distortion than is necessary. Holding weld sizes to print requirements will help avoid unnecessary distortion.

Obtain good fit-up: Poor fit-up, resulting in gaps and larger included angles for bevel preparations, means more weld metal is placed in the joint than is required, contributing to excessive distortion.

Use high-productivity, low-heat input welding procedures: Generally speaking, high-productivity welding procedures (those using high amperages and high travel speeds) result in a lower net heat input than low-productivity procedures. At first, high-amperage procedures may seem to be high-heat input procedures. However, for a given weld size, the high-amperage procedures are high travel-speed procedures. This will result in a decreased amount of heat-affected zone and reduced distortion.

Use clamps, strongbacks, or fixtures to restrict the amount of distortion: Any tooling or restraints that avoid rotation of the part will reduce the amount of distortion experienced. In addition, fixturing may be used to draw heat away, particularly if copper chill bars and clamps are used in the vicinity of the joint. The arc should never impinge on copper as this could cause cracking.

Use a well-planned welding sequence: A well-planned welding sequence is often helpful in balancing the shrinkage forces against each other.

Preset or precamber parts before welding: Parts to be joined may be preset or precambered before welding. When weld shrinkage causes distortion, the parts will be drawn back into proper alignment.

3.9 Special Welding Issues for Seismically Resistant Structures

3.9.1 Introduction and background

Steel is an inherently forgiving material, and welded construction is a highly efficient, direct method by which several members can be made to function as one. For statically loaded structures, the inherent ductility associated with steel allows the material to compensate for deficiencies in design, materials, and fabrication. For structures containing deficiencies in one or more of these three areas that are subject to cyclical loading, the repeated plastic (or inelastic) deformations can lead to fatigue failure when enough cycles are present. Fabrication of components such as bridges and crane rail supports is therefore more sensitive to deficiencies in design, materials, and fabrication than is fabrication of statically loaded structures. In a similar manner, welded structures subject to seismic loads are sensitive to these three issues as well. This section addresses those issues that are necessary to improve the seismic resistance of welded steel structures.

In addition to providing a concise summary of those items that should be considered in the design, fabrication, and erection of steel buildings subject to seismic loading, these items have been separated from the requirements in the body of this book in order to avoid the potential introduction of the conservative provisions of this chapter into requirements for structures subject to standard wind and gravity loads. Just as it would be inappropriate, and uneconomical, to impose on all buildings the requirements for bridge fabrication, so it would be undesirable to see these requirements for fabrication in seismic zones automatically transmitted to all structures. The cost increases without any obvious improvement in structure performance would be a disservice to the industry and ultimately the owners of these structures.

The principles presented in Chap. 5 generally apply to seismic construction as well, and the importance of conforming to these principles is even more significant for seismic construction. The very low levels of variable stress typically associated with structures subject to cyclical loading are significantly different from the inelastic loads imposed by an earthquake. Nevertheless, many of the details as they relate to weld backing, weld quality issues, and desirable details are relevant to seismically loaded structures. Chapter 3 provides insight into the available welding processes and the requirement to control quality, in both the shop and the field, when welding any structure. High-quality fabrication is essential for seismically resistant structures. The reader is encouraged to review these other sections, as no attempt has been made to replicate the contents of those chapters as they apply to seismically resistant structures.

Most fabrication work in the United States is performed in accordance with the *AWS D1.1 Structural Welding Code—Steel*. This code provides general requirements applicable to all welded structures. The D1.1 code does not, however, contain any requirements *unique* to seismic construction. Perhaps this will be done at a future date. The D1.1 code should be taken as the lower bound of acceptable fabrication practice, and the engineer can incorporate additional requirements into contract documents to ensure that the latest requirements for seismic resistance are employed. Section 8 of D1.1 contains the title "Statically Loaded Structures," and "Cyclically Loaded Structures." This is an improvement over the former title of "Dynamically Loaded Structures." Chapter provisions are specifically geared toward low-stress-range, high-cycle fatigue-type loading, not the high-stress-range, taw-cycle stress associated with seismic loading. It is generally recommended that Section 8 criteria be applied for seismic applications, and appropriate modifications be made through contract documents as new information becomes available.

At the time of the writing of this section, two significant earthquakes have occurred in the recent past. In both events, significant damage was experienced by welded steel structures, resulting in considerable commitments of resources to research. As a result of these events and the subsequent research, a better understanding of the expected behavior of various structural systems and details has been achieved. While many theories exist regarding various aspects of connection details, some remain unproved, and more testing is required. This section represents an accumulation of the best data available to date, and yet recognizes that information will be emerging in the next months and years that may render some of these recommendations incomplete or incorrect. Before adopting these provisions, the reader is cautioned to compare this information to the latest applicable specifications and the state of the art.

One of the best current sources of information is FEMA 267, "Interim Guidelines for Repair and Fabrication of Steel Moment Resisting Frames." This document is the result of the first phase of government-sponsored research performed by the consortium of the Structure of Engineers Association of California (SEAOC), Applied Technology Council (ATC), and California Universities for Research and Earthquake Engineering (CUREe), together known as SAC. The second phase of SAC research, also funded by FEMA, began in 1996 and was expected to continue through 1998. Additional information is expected from these studies.

The principles contained in this section are generally based upon well-founded engineering principles. However, not all of the recommendations based upon these principles have been subject to testing. Application of this information to a specific project is necessarily the responsibility of the engineer of record.

3.9.2 General review of welding engineering principles

For dynamically loaded structures, attention to detail is critical. This applies equally to high-cycle fatigue loading, short-duration abrupt impact loading, and seismic loading. The following constitutes a review of basic welding engineering principles that apply to all construction, but particularly to seismic applications.

Transfer of loads. All welds are not evenly loaded. This applies to weld groups that are subject to bending as well as those subject to variable loads along their length. A less obvious situation occurs when steels of different geometries are joined by welding. A rule of thumb is to assume the transfer of force takes place from one member, through the weld, to the member that lies parallel to the force that is applied. Several examples are illustrated in Fig. 3.23. For most simple static loading applications, redistribution of stress throughout the member accommodates the variable loading levels. For dynamically loaded members, however, this is an issue that must be carefully assessed in the design. The addition of stiffeners or continuity plates to column webs helps to unify the distribution of stress across the groove weld. Notice that the distribution of stress across the groove weld joining a beam to an I-shaped column is just opposite that of the same beam joined to a box column.

Minimize weld volumes. A good principle of welded design is to always use the least amount of weld metal possible for a given application. Not only does this have sound economic implications, but it reduces the level of residual stress in the connection. All hot expanded metals will shrink as they cool, inducing residual stresses in the connection. By reducing the volume of weld metal, these tendencies can be minimized. Details that will minimize weld volumes for groove welds generally involve minimum root openings, minimum included angles, and the use of double-sided joints. Taken to the extreme, however, these approaches may violate the principles outlined in "Provide Ample Access for Welding" section. By reducing the shrinkage stress, distortion and cracking tendencies can be minimized.

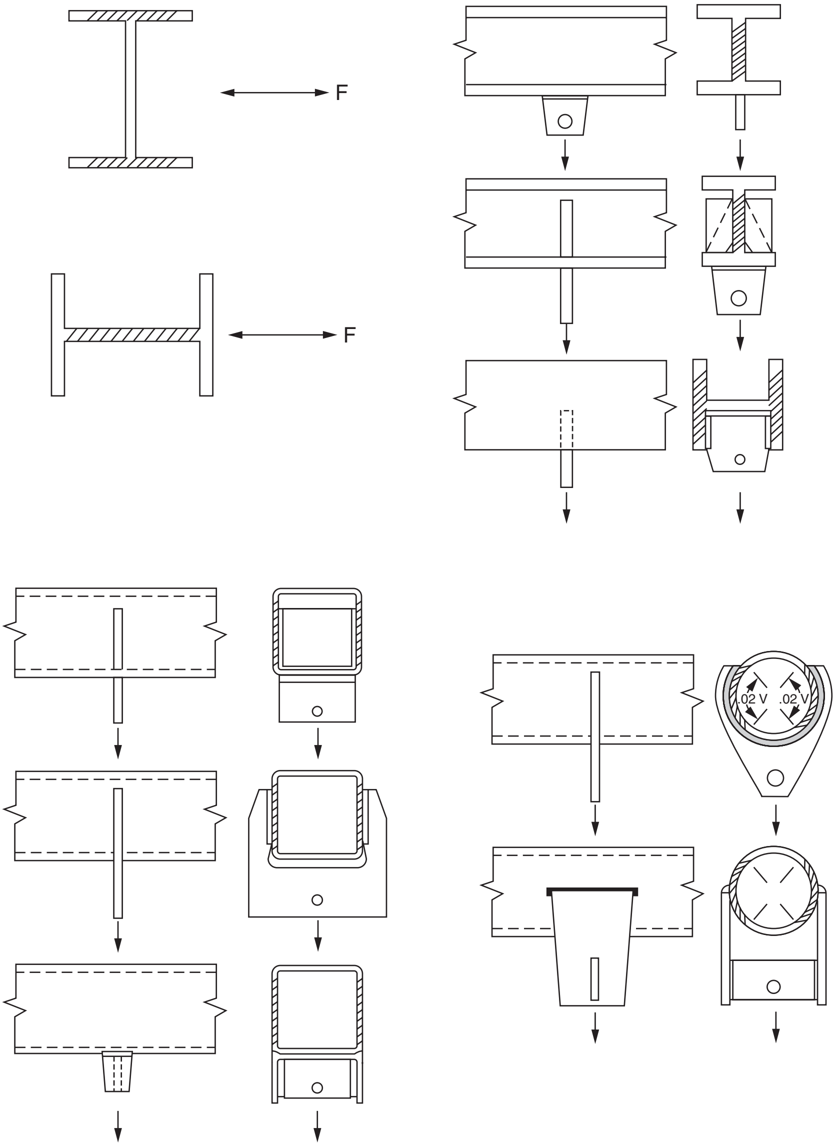


Figure 3.23 Examples of transfer of loads. (*Courtesy of The Lincoln Electric Company.*)

Recognize steel properties. Steel is not a perfectly isotropic material. The best mechanical properties usually are obtained in the same orientation in which the steel was originally rolled. This is called the X axis. Perpendicular to the X axis is the width of the steel, or the Y axis. Through the thickness, or the Z axis, the steel will exhibit the least amount of ductility, lowest strength, and lowest toughness properties. When possible, it is always desirable to allow the residual stresses of welding to elongate the steel in the X direction. Of particular concern are large welds placed on either side of the thickness of the steel where the weld shrinkage stress will act in the Z axis. This can result in lamellar tearing during the time of fabrication, or it can result in subsurface fracture during seismic loading.

Provide ample access for welding. It is essential that the weld joint design as well as the surrounding configuration of material offer adequate access and visibility for the welder and the welding equipment. If the operator cannot adequately observe the joint, weld quality will suffer. As a general rule, if the welder cannot see the joint, neither can the inspector. Weld quality will naturally suffer. It is important that adequate access is provided for the proper placement of the welding electrode with respect to the joint. This is a function of the welding process. Gas-shielded processes, for example, must have ample access for insertion of the shielding gas nozzle into the weld joint. Consideration of these issues has been incorporated into the prequalified groove weld details as listed in AWS D1.1. Overall access to the joint is a function of the configuration of the surrounding material. The designer and detailer should be aware of these general constraints in order to provide adequate access for high-quality fabrication.

No secondary members in welded design. A fundamental premise of welding design is that there are no secondary members. Anything that is joined by welding can, and will, transfer stress between joined materials. Segmented pieces of steel used for weld backing, for example, can result in a stress-concentration factor at the interface of the backing. Attachments that are merely tack-welded in place may become major load-carrying members, resulting in the initiation of fracture and propagation throughout the structure. These details must be considered in the design stage, and also controlled during fabrication and erection.

Residual stresses in welding. As hot expanded weld metal and the surrounding base metal cool to room temperature, they must shrink volumetrically. Under most welding conditions, this contraction is restrained or restricted by the surrounding material, which is relatively rigid and resists the shrinkage. This causes the weld to induce a residual stress pattern where the weld metal is in residual tension, and the surrounding base metal is in residual compression. The residual stress pattern is three-dimensional since the metal shrinks volumetrically. The residual stress distribution becomes more complicated when multiple-pass welding is performed. The final weld pass is always in residual tension, but previous weld beads that were formerly in tension will have compression induced by the subsequent passes.

For relatively flexible assemblages, these residual stresses induce distortion. As assemblages become more rigid, the same residual stresses can cause weld cracking problems, typically occurring near the time of fabrication. If distortion does not occur, or when cracking does not occur, the residual stresses do not relieve themselves but are "locked in." Residual stresses are considered to be at the yield point of the material involved. Because any area that is subject to residual tensile stress is surrounded by a region of residual compressive stress, there is no loss in overall capacity of as-welded structures. This does reduce the fatigue life for low-stress-range, high-cycle applications, which are different from seismic loading conditions.

Small welded assemblies can be thermally stress-relieved where the steel is heated to 1150°F, held for a predetermined length of time (typically 1 h/in of thickness), and allowed to return to room temperature. Residual stresses can be reduced by this method, but they are never totally eliminated. This type of approach is not practical for large assemblies, and care must be exercised to ensure that the components being stress-relieved have adequate support when at the elevated temperature where the yield strength and the modulus of elasticity are greatly reduced as opposed to room-temperature properties. For most structural applications, the residual stresses cause no particular problem to the performance of the system, and owing to the complications of stress-relief activities, welded structures commonly are used in the as-welded condition.

When loads are applied to as-welded structures, there is some redistribution or gradual decrease in the residual stress patterns. Typically called *shake-down,* the thermal expansion and contraction experienced by a typical structure as it goes through a climatic season, as well as initial service loads applied to the building, result in a gradual reduction in the residual stresses from welding.

These residual stresses should be considered in any structural application. On a macro level, they will affect the erector's overall sequence of assembly of a building. On a micro level, they will dictate the most appropriate weld bead sequencing in a particular groove-welded joint. For welding applications involving repair, control of residual stresses is particularly important since the degree of restraint associated with weld repair conditions is inevitably very high. Under these conditions, as well as applications involving heavy, highly restrained, very thick steel for new construction, the experience of a competent welding engineer can be helpful in avoiding the creation of unnecessarily high residual stresses, thus alleviating cracking tendencies.

Triaxial stresses and ductility. The commonly reported values for ductility of steel generally are obtained from uniaxial tensile coupons. The same degree of ductility cannot be achieved under biaxial or triaxial loading conditions. This is particularly significant since residual stresses are always present in any as-welded structure.

Flat position welding. Whenever possible, it is desirable to orient weld details so that the welding can be performed in the flat position, taking advantage of gravity which helps hold the molten weld metal in place. These welds are made with a lower requirement for operator skill, and at the higher deposition rates that correspond to more economical fabrication. This is not to say, however, that overhead welding should be avoided at all costs. An overhead weld may be advantageous if it allows for double-sided welding and a corresponding reduction in the weld volume. High-quality welds can be made in the vertical plane and with the welding consumables available today, can be made at an economical rate.

3.9.3 Unique aspects of seismically loaded welded structures

Demands on structural systems. Structures designed for seismic resistance are subject to extreme demands during earthquakes. By definition, any structure designed with an R_ω greater than unity will be loaded beyond the yield stress of the material. This is far more demanding than other anticipated types of loading. Because of the inherent ductility of the material, stress concentrations within a steel structure are gradually distributed by plastic deformation. If the materials have a moderate degree of notch toughness, this redistribution eliminates localized areas of high stress, whether due to design, material, or fabrication irregularities. For statically loaded structures, the redistribution of stresses is of little consequence. For cyclically loaded structures, repetition of this redistribution can lead to fatigue failure. In seismic loading, however, it is expected that portions of the structure will be loaded well beyond the elastic limit, resulting in plastic deformation. Localized areas of high stress will not simply be spread out over a larger region by plastic deformation. The resultant design, details, materials, fabrication, and erection must all be carefully controlled in order to resist these extremely demanding loading conditions.

Demand for ductility. Seismic designs have relied on "ductility" to protect the structural system during earthquakes. Unfortunately, much confusion exists regarding the measured property of ductility and steel, and ductility can be experienced in steel configured in various ways.

Because of the versatility of welding, there is the possibility of configuring the materials in ways in which ductility cannot be achieved. It is essential that a fundamental understanding of ductility be achieved in order to ensure ductile behavior in the steel in general, and in the welded connections in particular.

Unique capabilities of welded designs. Welding is the only method by which multiple pieces can be made to act in complete unison as one metallurgical system. Loads can be efficiently transferred through welded connections, and joints can be made to be 100% efficient. There are, however, no secondary members in welded designs. Weld backing, for example, participates in conducting forces through members, whether intended or not. The design versatility afforded by the welding process allows for configuring the material in less than optimum orientations. The constraints associated with other systems such as bolting or the use of steel castings provide other constraints that may force compromises for manufacturing but may simultaneously optimize the transfer of stress through the members. It is critical, therefore, that the engineer utilize designs that capitalize upon welding's advantages and minimize potential limitations.

Requirements for efficient welded structures. Five elements are present in any efficient welded structure:

1. Good overall design
2. Good details
3. Good materials
4. Good workmanship
5. Good inspection

Each element of the preceding list is important, and emphasis on one item will not overcome deficiencies in others. Both the Northridge and Kobe earthquakes have shown that most of the undesirable behavior is traceable to deficiencies in one or more of the preceding areas.

3.9.4 Design of seismically resistant welded structures

System options. Several systems are available to the designer in order to achieve seismic resistance, including the eccentrically braced frame (EBF), concentrically braced frame (CBF), special moment-resisting frames (SMRF), and base isolation. Of the four mentioned, only base isolation is expected to reduce demand on the structure.

The other three systems assume that at some point within the structure, plastic deformations will occur in members, thus absorbing seismic energy. In this section, no attempt is being made to compare the relative advantages of one system over another. Rather, the focus will be placed on the demands on welded connections' and member behavior with respect to the various systems that may be selected by the designer.

In the CBF system, the brace member is the one expected to be subject to inelastic deformations. The welded connections at the termination of a brace are subject to significant tension or compression loads, although rotation demands in the connections are fairly low. Designs of these connections are fairly straightforward, requiring the engineer to develop the capacity of the brace member in compression and tension. Recent experiences with CBF systems have reaffirmed the importance of the brace dimensions (*b/t* ratio), and the importance of good details in the connection itself. Problems seem to be associated with undersized welds, misplaced welds, missing welds, or welds of insufficient throat due to construction methods. In order to place the brace into the building frame, it is common to utilize a gusset plate welded into the corners of the frame. The brace is slit along its longitudinal axis and rotated into place. In order to maintain adequate dimensions for field assembly, it is necessary to oversize the slot in the tube as compared to the gusset. This results in natural gaps between the tube and the gusset plate. When this dimension increases, as illustrated in Fig. 3.24, it is important to consider the effect of the root opening on the strength of the fillet weld. The D1.1 code requires that, for gaps exceeding $^1/_{16}$ in, the weld leg size be increased by the amount of the gap. This ensures that a constant actual throat dimension is maintained.

EBFs and SMRFs are significantly different structural systems, but from a welding design point of view, there are principles that apply equally to both systems. It is possible to design an EBF so that the "link" consists simply of a rolled steel member. In Fig. 3.25, these examples are illustrated by the links designated as C1. In other EBF systems, however, the connection itself can be part of the link, as

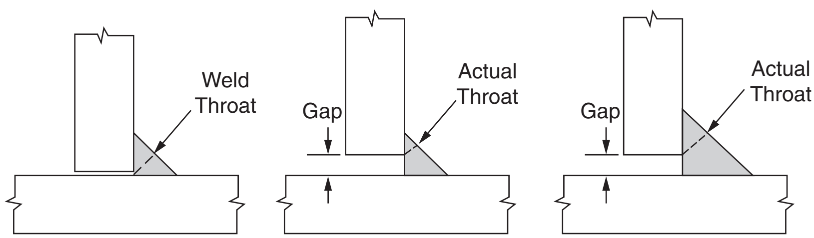


Figure 3.24 Effect of root openings (gaps) on fillet weld throat dimensions. (*Courtesy of The Lincoln Electric Company.*)

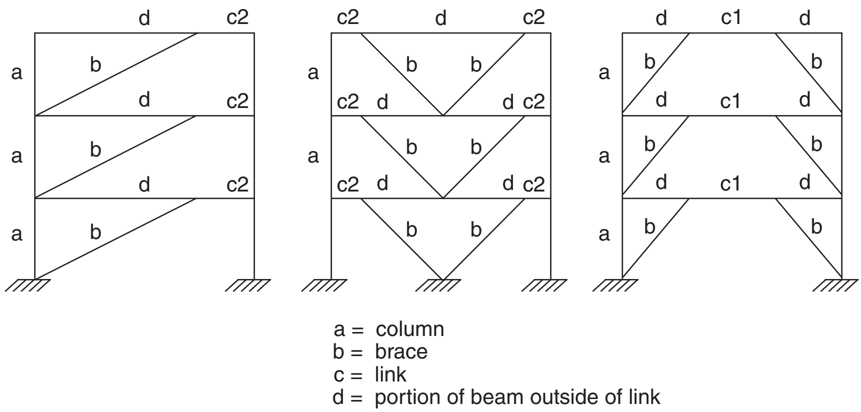
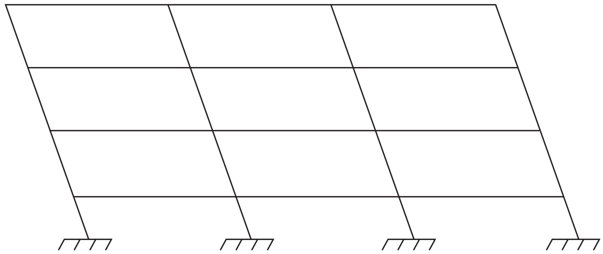


Figure 3.25 Examples of EBF systems. (*From "Seismic Provisions for Structural Steel Building," American Institute of Steel Construction, Inc., 1992.*)

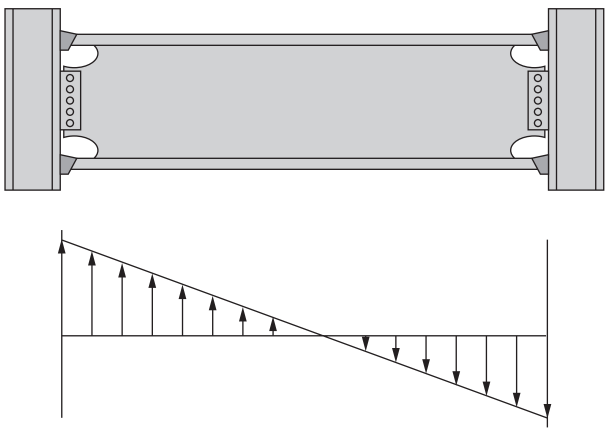
illustrated by C2. When an EBF system is designed by this method, the welded connections become critical since the expected loading on the connection is in the inelastic region. Much of the discussion under SMRF is applicable to these situations.

A common method applied to low-rise structures is the SMRF system. Advantages of this type of system include desirable architectural elements that leave the structure free of interrupting diagonal members. Extremely high demands for inelastic behavior in the connections are inherent to this system.

When subject to lateral displacements, the structure assumes a shape as shown in Fig. 3.26*a*, resulting in the moment diagram shown in Fig. 3.26*b*. Notice that the highest moments are applied at the connection. A plot of the section properties is schematically represented in Fig. 3.26*c*. Section properties are at their lowest value at the column face, owing to the weld access holes that permit the deposition of the complete joint-penetration (CJP) beam flange to column flange welds. These section properties may be further reduced by the deletion of the beam web from the calculation of section properties. This is a reasonable assumption when the beam web to column shear tab is connected by the means of high-strength bolts. Greater capacity is achieved when the beam web is directly welded to the column flange with a complete joint-penetration groove weld. The section properties at the end of the beam are least, precisely an area where the moment levels are the greatest. This naturally leads to the highest level of stresses. A plot of stress distribution is shown in Fig. 3.26*d*. The weld is therefore in the area of highest stress, making it critical to the performance of the connection. Details in either SMRF systems or EBF systems that place this type of demand on the weld require careful scrutiny.

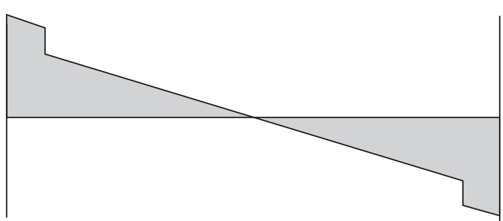


a. SMRF systems subject to lateral displacements.



b. Moment diagram of SMRF subject to lateral displacements.

c. Section properties of SMRF subject to lateral displacements.



d. Stress distribution of SMRF subject to lateral displacements.

Figure 3.26 Analysis of SMRF behavior. (*Courtesy of The Lincoln Electric Company.*)

Ductile hinges in connections. The fundamental premise regarding the special moment-resisting frame (SMRF) is that plastic hinges will form in the beams, absorbing seismically induced energies by inelastically stretching and deforming the steel. The connection is not expected to break. Following the Northridge earthquake, there was little or no evidence of hinge formation. Instead, the connections or portions of the connection experienced brittle fracture, inconsistent with expected and essential behavior. Most of the ductility data are obtained from smooth, slowly loaded, uniaxially loaded tensile specimens that are free to neck down. If a notch is placed in the specimen, perpendicular to the applied load, the specimen will be unable to exhibit its normal ductility, usually measured as elongation. The presence of notchlike conditions in the Northridge connections decreased the ductile behavior.

Initial research on SMRF connections conducted in the summer of 1994 attempted to eliminate the issues of notchlike conditions in the test specimens by removing weld backing and weld tabs, and controlling weld soundness. Even with these changes, "brittle" fractures occurred when the standard details were tested. The testing program then evaluated several modified details with short cover plates, with better success. The reason for these differences can be explained analytically.

Referring to Fig. 3.27, the material at point A, whether it be weld metal or base metal, cannot exhibit the ductility of a simple tension test. Ductility can take place only if the material can slip in shear along numerous slip planes. That is, it must be free to neck down.

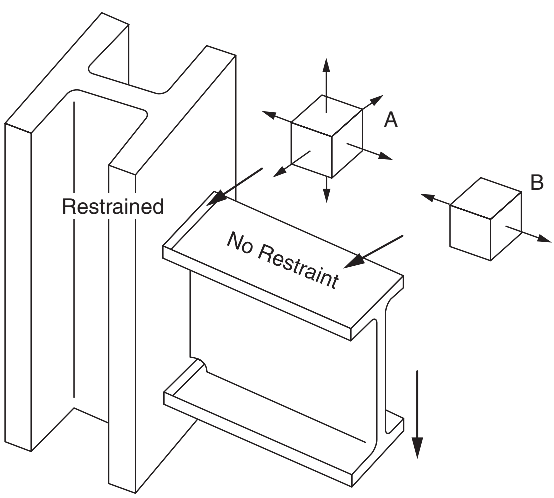


Figure 3.27 Regions to be analyzed relative to potential for ductile behavior. (*Courtesy of The Lincoln Electric Company.*)

Four conditions are required for ductility:

1. There must be a shear stress (τ) component resulting from the given load condition.
2. This shear stress must exceed its critical value by a reasonable amount. The more it exceeds this value, the greater will be the resulting ductility.
3. The plastic shear strain resulting from this shear stress must act in the direction which will relieve the particular stress which can cause cracking.
4. There must be sufficient unrestrained length of the member to permit "necking down."

If conditions (1) and (2) are not met, there will be no apparent ductility and no yielding. The stress will simply build up to the ultimate tensile strength with little or no plastic energy absorbed. This condition is called brittle failure.

Figure 3.27 shows two regions in question. Point *A* is at the weld joining the beam flange to the face of the column flange. Here there is restraint against strain (movement) across the width of the beam flange (ε_1) as well as through the thickness of the beam flange (ε_2). Point *B* is along the length of the beam flange away from the connecting weld. There is no restraint across the width of the flange or through its thickness.

In most strength of materials texts, the following equations can be found:

$$\varepsilon_3 = \frac{1}{E}(\sigma_3 - \mu\sigma_2 - \mu\sigma_1) \tag{3.1a}$$

$$\varepsilon_2 = \frac{1}{E}(-\mu\sigma_3 + \sigma_2 - \mu\sigma_1) \tag{3.1b}$$

$$\varepsilon_1 = \frac{1}{E}(-\mu\sigma_3 - \mu\sigma_2 + \sigma_1) \tag{3.1c}$$

It can be shown that

$$\sigma_1 = \frac{E[\mu\varepsilon_3 + \mu\varepsilon_2 + (1-\mu)\varepsilon_1]}{(1+\mu)(1-2\mu)} \tag{3.2a}$$

$$\sigma_2 = \frac{E[\mu\varepsilon_3 + (1-\mu)\varepsilon_2 + \mu\varepsilon_1]}{(1+\mu)(1-2\mu)} \tag{3.2b}$$

$$\sigma_3 = \frac{E[(1-\mu)\varepsilon_3 + \mu\varepsilon_2 + \mu\varepsilon_1]}{(1+\mu)(1-2\mu)} \tag{3.2c}$$

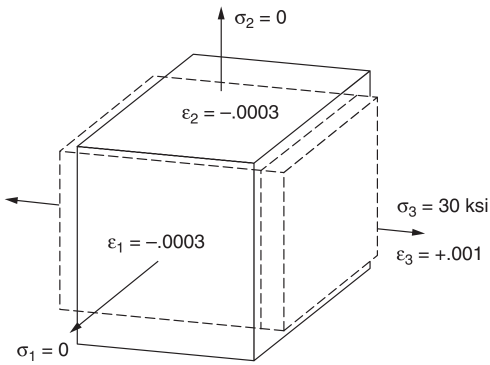


Figure 3.28 Unit cube showing applied stress from Fig. 3.27. (*Courtesy of The Lincoln Electric Company.*)

Consider the unit cube in Fig. 3.28. It is an element of the beam flange from point *B* in Fig. 3.27. The applied force due to the moment is σ_3. There is no restraint against the longitudinal stress of 30 ksi in the flange, so $\sigma_1 = \sigma_2 = 0$. Using Poisson's ratio of $\mu = 0.3$ for steel, Eqs. (3.1*a*) to (3.1*c*) yield the following for $\sigma_3 = 30$ ksi:

$$\varepsilon_3 = +0.001 \text{ in/in}$$
$$\varepsilon_2 = -0.0003 \text{ in/in}$$
$$\varepsilon_1 = -0.0003 \text{ in/in}$$

From Eqs. (3.2*a*) to (3.2*c*), it is found that

$$\sigma_1 = 0 \text{ ksi}$$
$$\sigma_2 = 0 \text{ ksi}$$
$$\sigma_3 = 30 \text{ ksi}$$

These stresses are plotted in Fig. 3.29 as a dotted circle. The larger solid-line circle is for a stress of 70 ksi or ultimate tensile stress.

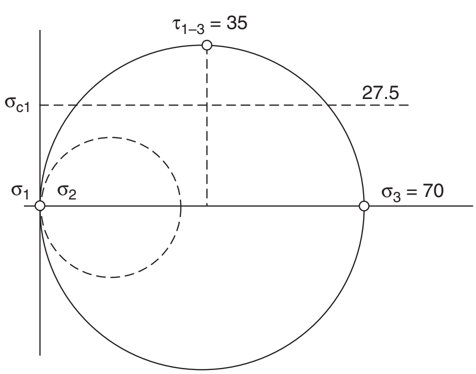


Figure 3.29 A plot of the tensile stress and shear stress from Fig. 3.27. (*Courtesy of The Lincoln Electric Company.*)

The resulting maximum shear stresses τ_{1-3} and τ_{2-3} are the radii of these two circles, or 35 ksi. The ratio of shear to tensile stress is 0.5. Figure 3.30 plots this as line *B*. Notice at a yield point of 55 ksi, the critical shear value is one-half of this, or 27.5 ksi. When this critical shear stress is reached, plastic straining or movement takes place and ductile behavior will result up to the ultimate tensile strength, here 70 ksi. Figure 3.31 shows a predicated stress-strain curve indicating ample ductility.

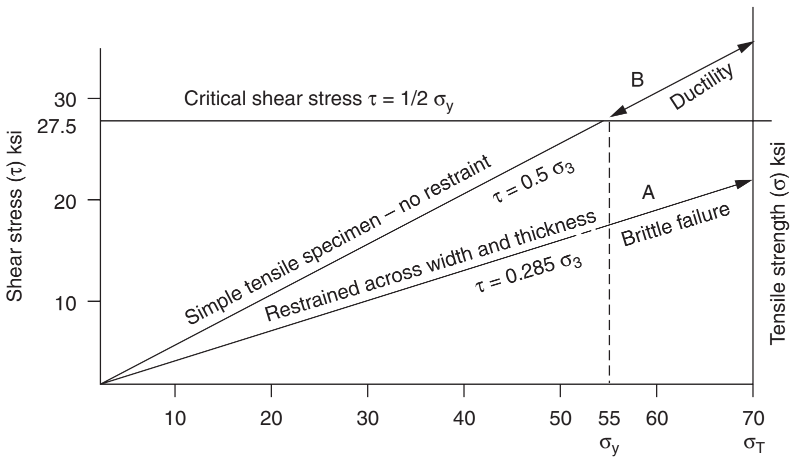


Figure 3.30 The ratio of shear to tensile stress. (*Courtesy of The Lincoln Electric Company.*)

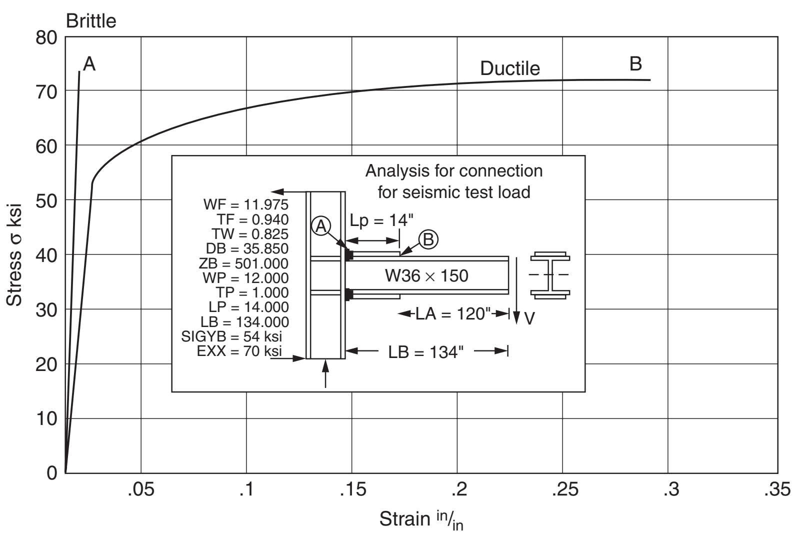


Figure 3.31 Stress-strain curve. (*Courtesy of The Lincoln Electric Company.*)

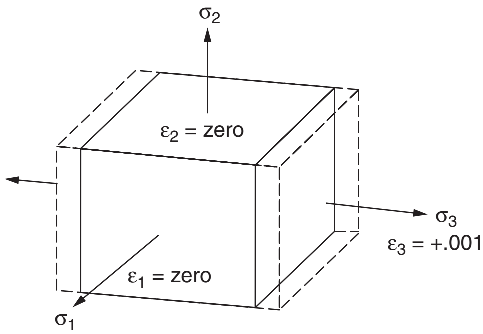


Figure 3.32 The highly restrained region at the junction of the beam and column flange as shown in Fig. 3.27. (*Courtesy of The Lincoln Electric Company.*)

Figure 3.32 shows an element from point *A* (Fig. 3.27) at the junction of the beam and column flange. Whether weld metal or the material in the column or beam is considered makes little difference. This region is highly restrained. Suppose it is assumed:

$\varepsilon_3 = +0.001$ in/in (as before)
$\varepsilon_2 = 0$ (since it is highly restrained with little strain possible)
$\varepsilon_1 = 0$

Then from the given equations, the following stresses are found:

$$\sigma_1 = 17.31 \text{ ksi}$$
$$\sigma_2 = 17.31 \text{ ksi}$$
$$\sigma_3 = 40.38 \text{ ksi}$$

In Fig. 3.33, these stresses are plotted as a dotted circle.

If these stresses are increased to the ultimate tensile strength, it is found that

$$\sigma_1 = 30.0 \text{ ksi}$$
$$\sigma_2 = 30.0 \text{ ksi}$$
$$\sigma_3 = 70.0 \text{ ksi}$$

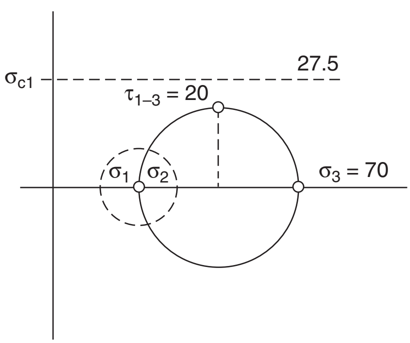


Figure 3.33 A plot of the tensile stress and shear stress from Fig. 3.32. (*Courtesy of The Lincoln Electric Company.*)

The solid-line circle in Fig. 3.33 is a plot of stresses for this condition. The maximum shear stresses are $\tau_{1-3} = \tau_{2-3} = 20$ ksi. Notice that these are less than the critical shear stress (27.5 ksi) so no plastic movement, or ductility, would be expected.

In this case, the ratio of shear to tensile stress is 0.286. In Fig. 3.30, this condition is plotted as line *A*. Notice it never exceeds the value of the critical shear stress (27.5 ksi); therefore, there will be no plastic strain or movement, and it will behave as a brittle material. Figure 3.31 shows a predicated stress-strain curve going upward as a straight line *A* (elastic) until the ultimate tensile stress is reached in a brittle manner with no energy absorbed plastically. It would therefore be expected that, at the column face or in the weld where high restraint exists, little ductility would be exhibited. This is where "brittle" fractures have occurred, both in actual Northridge buildings and in laboratory test specimens.

In the SMRF system, the greatest moment (due to lateral forces) will occur at the column face. This moment must be resisted by the beam's section properties, which are lowest at the column face due to weld access holes. Thus the highest stresses occur at this point, the point where analysis shows ductility to be impossible.

In Fig. 3.27, material at point *B* was expected to behave as shown in Fig. 3.29, and as line *B* in Fig. 3.30, and curve *B* in Fig. 3.31; that is, with ample ductility. It is essential that plastic hinges be forced to occur in this region.

Several post-Northridge designs have employed details that facilitate use of this potential ductility. Consider the cover-plated design illustrated in Fig. 3.34. Notice that this detail accomplishes two important purposes: first, the stress level at point *A* is reduced as a

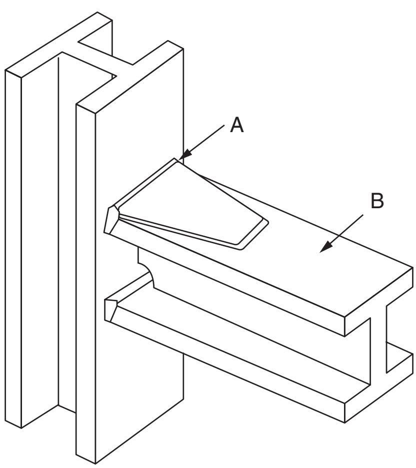


Figure 3.34 Cover-plated detail takes advantage of the region where ductility is possible. (*Courtesy of The Lincoln Electric Company.*)

result of the increased cross section at the weld. This region, incapable of ductility, must be kept below the critical tensile stress, and the increase in area accomplishes this goal. Second, and most important, the most highly stressed region is now at point *B,* the region of the beam that is capable of exhibiting ductility. Assuming good workmanship with no defects or stress raisers, the real success of this connection will depend upon getting the adjacent beam to bend plastically before this critical section cracks. The way in which a designer selects structural details under particular load conditions greatly influences whether the condition provides enough shear stress component so that the critical shear value may be exceeded first, producing sufficient plastic movement before the critical normal stress value is exceeded. This will result in a ductile detail and minimize the chances of cracking.

Details of welded connections. There are no secondary members in welded construction. Any material connected by a weld participates in the structural system—positively or negatively. Unexpected load paths can be developed by the unintentional metallurgical path that results from the one-component system created by welding. This must be considered in all phases of a welded steel project but is particularly significant in detailing.

Weld backing. Weld backing consists of auxiliary pieces of material used to support liquid weld puddles. The backing can be either temporary or permanent. Permanent backing consists of a steel member of similar composition that is fused in place by the weld. The D1.1 code requires that backing, if used, be continuous for the length of the joint, free of interruptions that would create stress-concentration factors. Segments of backing can be made continuous if complete joint-penetration (CJP) groove welds join the various segments of backing. It is essential that these splices be completely fused across the backing cross section.

Weld tabs. Weld tabs are auxiliary pieces of metal on which the welding arc may be initiated or terminated. For statically loaded structures, these are typically left in place. For seismic construction, it is recommended that weld tabs be removed from critical connections that are subject to inelastic loading. It is in the region of these weld tabs that metal of questionable integrity may be deposited. After removal, the end of the weld joint can be visually inspected for integrity.

Weld tab removal is probably most significant on beam-to-column connections where the beam flange width is less than the column flange width. It is reasonable to expect that stress flow would take place through the left-in-place weld tab. In contrast, for butt splices where the same width of material is joined, weld tabs that extend beyond the width of the joint would not be expected to carry significant stress levels, making weld tab removal less important. Tab removal from continuity plate welds is probably not justified.

The presence of weld tabs left-in-place is probably most significant for beam-to-column connections where columns are box shapes. The natural stress distribution under these conditions causes the ends of the groove weld between the beam and column to be loaded to the greatest level, the same region as would contain the weld tab. Just the opposite situation exists when columns are composed of I-shaped members. The center of the weld is loaded most severely, and the areas in which the weld tabs would be located have the lowest stress level. For welds subject to high levels of stress, however, weld tab removal is recommended.

Welds in combination with bolts. Welds provide a continuous metallurgical path that relies upon the internal metallurgical structure of the fused metal to provide continuity and strength. Mechanical fasteners such as rivets and bolts rely on friction, shear of the fastening element, or bearing of the joint material to provide for transfer of loads between members. When welds are combined with bolts, caution must be exercised in assigning load-carrying capacity to each joining method.

Traditionally it has been assumed that welds that are used in conjunction with bolts should be designed to carry the full load, assuming that the mechanical fasteners have no load-carrying capacity until the weld fails. With the development of high-strength fasteners, it has been assumed that loads can be shared equally between welds and fasteners. This has led to connection details which employ both joining systems. In particular, the welded flange, bolted web detail used for many beam-to-column connections in special moment-resisting frames (SMRF) assumes that the bolted web is equally able to share loads with the welded flanges. While most analysis suggests that vertical loads are transferred through the shear tab connection (bolted) and moments are transferred through the flanges (welded), the web does have some moment capacity. Depending on the particular rolled shape involved, the moment capacity of the web can be significant. Testing of specimens with the welded web detail, as compared to the bolted web detail, generally has yielded improved performance results. This has drawn into question the adequacy of the assumption of high-strength bolts sharing loads with welds when subject to inelastic loading. Research performed after the Northridge earthquake provides further evidence that the previously accepted assumptions may have been inadequate. This is not to suggest that bolted connections cannot be used in conjunction with welded connections. However, previous design rules regarding the capacity of bolted connections need to be reexamined. This may necessitate additional fasteners or larger sizes of shear tabs (both in thickness and in width). The rules regarding the addition of supplemental fillet welds on shear tabs, currently a function of the ratio of Z_f/Z, are very likely also inadequate and will require revision.

Until further research is done, the conservative approach is probably to utilize welded web details. This does not preclude the use of a bolted shear tab for erection purposes but would rely on welds as a singular element connecting the web to the column.

Some of the alternate designs that have been contemplated after the Northridge earthquake (see "Cover-Plated Designs") increase the moment capacity of the connection, reducing the requirement for the web to transfer moment. These details are probably less sensitive to the degree of interaction between welds and bolts.

Weld access holes. Weld access holes are openings in the web of a member that permit the welder to gain necessary access for the deposition of quality weld metal in a flange connection. Colloquially known as "rat holes," these openings also limit the interaction of residual stress fields present when a weld is completed. Poorly made weld access holes, as well as improperly sized holes, can limit the performance of a connection during seismic loading.

Consider the beam-to-column connection illustrated in Fig. 3.35. A welded web connection has been assumed. As the flange groove weld

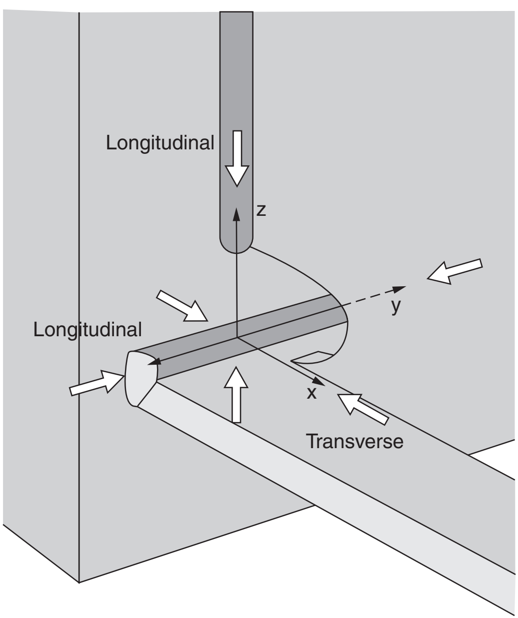


Figure 3.35 A generous weld access hole in this beam-to-column connection provides resistance to cracking. (*Courtesy of The Lincoln Electric Company.*)

shrinks volumetrically, a residual stress field will develop perpendicular to the longitudinal axis of the weld, as illustrated in direction X in the figure. Simultaneously, as the groove weld shrinks longitudinally, a residual stress pattern is established along the length of the weld, designated as direction Y. When the web weld is made, the longitudinal shrinkage of this weld results in a stress pattern in the Z direction. At the intersection of the web and flange of the beam with the face of the column, these three residual stress patterns meet at a point. When steel is loaded in all three orthogonal directions simultaneously, it is impossible for the most ductile steel to exhibit ductility. At the intersection of these three welds, cracking tendencies would be significant. By providing a generous weld access hole, however, the interaction of the Z-axis stress field and the biaxial (X and Y) stress field is physically interrupted. This increases the resistance to cracking during fabrication.

Residual compression in one axis can combine with residual tension in one or more axes, resulting in an increase in the ductility that will be observed. Ideally, weld access holes should be placed in areas where at least one residual compressive stress is present. Consider the longitudinal residual stress of the groove weld in the Y axis. While residual tension stresses are present in the weld itself, the weld is surrounded by a region of residual compressive stress that counteracts the tensile component. When weld access holes terminate in this area of residual compression, the connection will exhibit enhanced ductility.

For these reasons, AISC and AWS both have minimum weld access hole sizes that must be achieved. These minimums fit into two categories: minimums that must be obtained under all conditions, and minimum dimensions that are a function of the web thickness of the materials being joined. The absolute minimums are a function of the welding processes and good workmanship requirements. The web thickness-dependent dimensions reflect upon the geometric influence of member size upon the level of residual stresses that will be present. With larger member sizes, and correspondingly increased residual stresses, larger weld access holes are somewhat self-compensating.

Initially, the purpose of weld access holes was to provide adequate access for the welder to deposit quality weld metal. Failure to supply the welder with ample visibility of, and access to, the joint will naturally diminish quality. For beam-to-column connections, welding through the access hole when making the bottom beam flange weld has been demonstrated to be a particularly challenging task. Adequate sizing is partially dependent on the weld process being used. For SMAW, the relatively small diameter of the electrodes, coupled with their significant length, allows for easy access into this region. FCAW-ss may employ longer electrode extensions which somewhat duplicate the access flexibility of SMAW. Since FCAW-ss does not utilize a shielding gas, this process is unencumbered with gas nozzles that further restrict visibility. If FCAW-gs is used, however, allowance must be made for the operator to be able to place the welding gun, complete with a gas delivery nozzle, into the joint. For these reasons, weld access holes may have to be larger than the minimum dimensions prescribed in the applicable codes. It is usually best to let the fabricator's detailer select appropriately sized access holes for the particular welding processes to be used.

The quality of weld access holes is an important variable that affects both resistance to fabrication-related cracking as well as resistance to cracking that may result from seismic events. Access holes usually are cut into the steel by the use of a thermal cutting process, either oxy-fuel cutting (frequently called *burning*) or plasma arc cutting. Both processes rely on heating the steel to a high temperature and removing the heated material by pressurized gases. In the case of oxy-fuel cutting, oxidation of the steel is a key ingredient in this process. In either process, the steel on either side of the cut (called the *kerf*) has been heated to an elevated temperature and rapidly cooled. In the case of oxy-fuel cutting, the surface may be enriched with carbon. For plasma cut surfaces, metallic compounds of oxygen and nitrogen may be present on this surface. The resultant surface may be hard and crack-sensitive, depending on the combinations of the cutting procedure, base metal chemistry, and thickness of the materials involved. Under some conditions, the surface may contain small cracks. These cracks can be the points of stress amplification that cause further cracking during fabrication or during seismic events.

Nicks or gouges may be introduced during the cutting process, particularly when the cutting torch is manually propelled during the formation of the access hole. These nicks may act as stress-amplification points, increasing the possibility of cracking.

To decrease the likelihood of notches and/or microcracks on thermally cut surfaces, AISC has specific provisions that are required for making access holes in heavy group 4 and 5 rolled shapes. These provisions include the need for a preheat before cutting (to minimize the possibility of the formation of hard, crack-sensitive microstructures), requirements for grinding of these surfaces (to provide for a smooth contour, and to eliminate cracks and gouges as well as hard material that may be present), and inspection of these surfaces with magnetic-particle (MT) or dye-penetrant (PT) inspection (to verify a crack-free surface). Whether these requirements are necessary for all connections that may be subject to seismic energies is unknown at this time. However, for connection details that impose high levels of stress on the connection, and specifically those that demand inelastic performance, it is apparent that every detail in this region, including weld access hole geometry and quality, is a critical variable. Some cracking initiated from weld access holes in the Northridge earthquake. Whether this was the result of preexistent cracks that occurred during flange cutting, or the result of strains that were induced by the shrinkage of the welds during fabrication and/or erection, or simply the concentration of seismically induced forces that were amplified in these regions, is not known at this time. It does highlight the importance, however, of paying attention to all construction details, including weld access holes.

3.9.5 Materials

Base metal. Base metal properties are significant in any type of steel construction but particularly in structures subject to seismic loading. While most static designs do not require loading beyond the yield strength of the material, seismically resistant structures depend on acceptable material behavior beyond the elastic limit. Although most static designs attempt to avoid yielding, the basic premise of seismic design is to absorb seismic energies through yielding of the material. For static design, additional yield strength capacity in the steel may be desirable. For applications where yielding is the desired method for achieving energy absorption, higher than expected yield strengths have a dramatic negative effect on some designs. This is particularly important as it relates to connections, both bolted and welded.

Figure 3.36 illustrates five material zones that occur near the groove weld in a beam-to-column connection. This is the standard

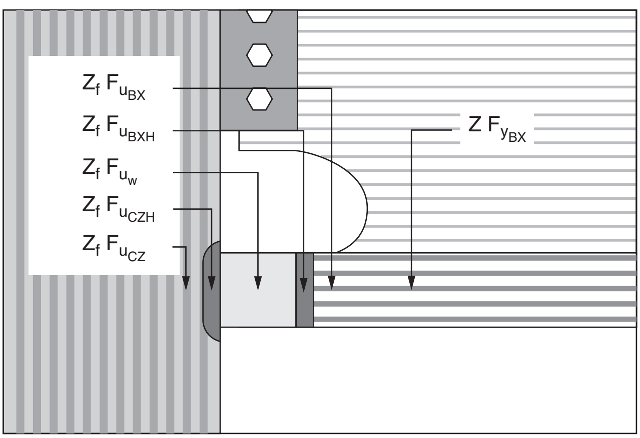


Figure 3.36 Five material zones that occur near the groove weld in a beam-to-column connection. (*Courtesy of The Lincoln Electric Company.*)

connection detail used for SMRF systems, at least prior to the Northridge earthquake. If it is assumed that the web is incapable of transferring any moment (a simple assumption, but probably justified by the lack of confidence in the ability of welds to share loads with bolts), it is critical that the plastic section modulus of the flanges Z_f, times the tensile strength be greater than the entire plastic section property Z times the yield strength in the beam. All five material properties must be considered in order for the connection to behave satisfactorily.

Existing ASTM specifications for most structural steels do not place an upper limit on the yield strength but specify only a minimum acceptable value. For example, for ASTM A36 steel, the minimum acceptable yield strength is 36 ksi.

This requirement precludes a steel that has a yield strength of 35.5 ksi as being acceptable but does nothing to prohibit the delivery of a 60-ksi steel. The tensile strength range is specified as 58 to 80 ksi. While A36 is commonly specified for beams, columns are typically specified to be of ASTM A572 grade 50. With a 50-ksi minimum yield strength, and a minimum tensile strength of 65 ksi, many designers were left with the false impression that the yield strength of the beam could naturally be less than that of the column. Owing to the specification requirements, it is possible to produce steel that meets the requirements of both A36 and A572 grade 50. This material has been commercially promoted as "dual certified" material. In actual fact, regardless of what the material is called, it is critical for the connection illustrated in Fig. 3.36 to have controls on material properties that are more rigorous than the current ASTM standards impose.

Much of the focus has related to the beam yield-to-tensile ratio, commonly denoted as F_y/F_u. This is often compared to the ratio of Z_f/Z, with the desired relationship F_y/F_u being

$$\frac{Z_f}{Z} > \frac{F_y}{F_u}$$

This suggests that not only is F_y (yield strength) important, but the ratio is important as well. For rolled W-shapes, Z_f/Z ranges from 0.6 to 0.9. Based on ASTM *minimum* specified properties, F_yF_u is as follows:

A36	0.62
A572Gr50	0.77

However, when actual properties of the steel are used, this ratio may increase. In the case of one Northridge damaged building, mill test reports indicated the ratio to be 0.83.

ASTM steel specifications need additional controls that will limit the upper value of acceptable yield strengths for materials as well as the ratio of F_y/F_u. A new ASTM specification has been proposed that will address these issues, although its approval will probably not be achieved before 1997.

In Fig. 3.36, five zones have been identified in the area of the connection, with the sixth material property being located in the beam. Thus far, only two have been discussed, namely, the beam yield strength and the beam ultimate strength. As shown in the figure, these are designated with the subscript X to indicate that these are the properties in the orientation of the longitudinal axis of the beam.

The properties of interest with respect to the column are oriented in the column Z axis. This is the direction which will exhibit the least desirable mechanical properties. Current ASTM specifications do not require measurement of properties in this orientation. While there are ASTM standards for the measurement of through-thickness properties (ASTM 770), these are not normally applied for structural applications. It is this through-thickness strength, however, that is important to the performance of the connection.

The weld metal properties are also very important, as indicated by the designation F_{u_w} in Fig. 3.36. These are discussed in "Weld Metal Properties."

On either side of the weld are two heat-affected zones (HAZ), one located on the column side of the weld and the other located on the beam side. These properties are also important to the performance of the connection and are discussed in "Heat-Affected Zones."

Notch toughness is defined as the ability of a material to resist propagation of a preexisting cracklike flaw while under tensile stress. Pre-Northridge specifications did not have required notch toughness for either base materials or weld metals. When high loads are applied, and when notchlike details or imperfections exist, notch toughness is the material property that resists brittle propagation from that discontinuity. Rolled shapes routinely produced today, specifically for lighter-weight shapes in the group 1, 2, and 3 category, generally are able to deliver a minimum notch toughness of 15 ft lb at 40°F. This is probably adequate toughness, although additional research should be performed in this area.

For heavy columns made of group 4 and 5 shapes, this level of notch loughness may not be routinely achieved in standard production. After Northridge, many engineers began to specify the supplemental requirements for notch toughness that are invoked by AISC specifications for welded tension splices in jumbo sections (group 4 and 5 rolled shapes). This requirement for 20 ft lb at 70°F is obtained from a Charpy specimen extracted from the web-flange interface, an area expected to have the lowest toughness in the cross section of the shape. Since columns are not designed as tension members under most conditions, this requirement would not automatically be applied for column applications. However, as an interim specification, it seems to be a reasonable approach to ensure minimum levels of notch toughness for heavy columns.

Weld metal properties. Four properties of interest typically are applied to weld metal: yield strength, tensile strength, toughness, and elongation. These properties are generally obtained from data on the particular filler metal that will be employed to make the connection. The American Welding Society (AWS) filler metal classification system contains a "coding" that defines the minimum acceptable properties for the weld metal when deposited under very specific conditions. Most 70 series electrodes (for example, E7018, E70T-1, E70T-6) have a minimum specified yield strength of 58 ksi and a minimum tensile strength of 70 ksi. As in the specifications for steel, there are no upper limits on the yield strength. However, in welded design, it is generally assumed that the weld metal properties will exceed those of the base metal, and any yielding that would occur in the connection should be concentrated in the base metal, not in the weld metal, since the base metal is assumed to be more homogeneous and more likely to be free of discontinuities that may be contained within the weld. Most of the commercially available filler metals today have a 70 classification, which exceeds the minimum specified strength properties of the commonly used A36 and A572 grade 50.

These weld metal properties are obtained under very specific testing conditions that are prescribed by the AWS A5 filler metal specifications. Weld metal properties are a function of a variety of variables, including preheat and interpass temperatures, welding parameters, base metal composition, and joint design. Deviations in these conditions from those obtained for the test welds may result in differences in mechanical properties. Most of these changes will result in an increase in yield and tensile strength, along with a corresponding decrease in elongation and, in general, a decrease in toughness. When weld metal properties exceed those of the base metal, and when the connection is loaded into the inelastic range, plastic deformations would be expected to occur in the base metal, not in the weld metal itself. The increase in the strength of the weld metal compensates for the loss in ductility. The general trend to strength levels higher than those obtained under the testing conditions is generally of little consequence in actual fabrication.

There are conditions that may result in lower levels of strength being obtained, and the Northridge earthquake experience revealed that this may be more commonplace and more significant than originally thought. The interpass temperature is the temperature of the steel when the arc is initiated for subsequent welding. There are two aspects to the interpass temperature: the minimum level, which should always be the minimum preheat temperature, and the maximum level, beyond which welding should not be performed. Because of the relatively short length of beam-to-column flange welds, it is possible for a welder to continue welding at a pace that will allow the temperature of the steel at the connection to increase to unacceptably high levels. After one or two weld passes, this temperature may approach the 1000°F range. Under these conditions, a marked decrease in the strength of the weld deposit may occur.

Although it would be unexpected to see the strength drop below the minimum specified property for A572 grade 50 steel, it may fall below the typical strength of the weld deposit made under more controlled conditions. The restraint associated with the geometry at the beam-to-column junction does not encourage yielding, so the decrease in uniaxial yield strength may have less significance than the decrease in tensile capacity.

Much emphasis has been placed on elongation of materials, but as discussed under sections "Demands on Structural Systems" and "Demand for Ductility," geometric constraints on ductility would generally preclude welds from being able to deform, regardless of their uniaxial elongation properties.

Weld metal toughness is an area of particular interest in the post-Northridge specifications. Previous specifications did not have any requirement for minimum notch toughness levels in the weld deposits, allowing for the use of filler metals that have no minimum specified requirements. For connections that are subject to inelastic loading, it seems apparent that minimum levels of notch toughness must be specified. The actual limits on notch toughness have not been experimentally determined. With the AWS filler metal classifications in effect in 1996, they are either classified as having no minimum specified notch toughness, or with properties of 20 ft lb at a temperature of 0°F or lower. As an interim specification, 20 ft lb at 0°F or lower has been recommended. However, this has been based upon availability, not on an analysis of actual requirements. It is expected that actual requirements will be less demanding, and once these requirements are determined, new filler metals will be developed that will meet the appropriate requirements. It should be recognized that the more demanding notch toughness requirements impose several undesirable consequences upon fabrication, including increased cost of materials, lower rates of fabrication (deposition rates), less operator appeal, and greater difficulty in obtaining sound weld deposits. Therefore, ultraconservative requirements imposed "just to be safe" may be practically and economically unacceptable. Research will be conducted to determine actual toughness requirements.

Heat-affected zones. As illustrated in Fig. 3.36, the base metal heat-affected zones represent material that may affect connection performance as well. The *heat-affected zone* (HAZ) is defined as that base metal which has been thermally changed due to the energy introduced into it by the welding process. In the small region immediately adjacent to the weld, the base metal has gone through a different thermal history than the rest of the base material. For most hot-rolled steels, the area of concern is a HAZ that is cooled too rapidly, resulting in a hardened heat-affected zone. For quenched-and-tempered steels, the HAZ may be cooled too slowly, resulting in a softening of the area. In columns, the HAZ of interest is the Z direction properties immediately adjacent to the groove weld. For the beam, these are oriented in the X direction.

Heat-affected zone properties are a function of base metal chemistry and specific welding procedures. Steel makers consider HAZ properties when developing a specific steel composition, and for quenched-and-tempered steels, guidelines are available from the steel producer indicating what precautions must be taken during welding to preclude the formation of undesirable heat-affected zones. The primary welding variables that affect HAZ properties are the preheat and interpass temperatures (both minimum and maximum), and the heat input of welding. Excessively high heat input can negatively affect HAZ properties by causing softening in these areas. Excessively low heat input can result in hardening of the HAZ.

Weld metal properties may be negatively affected by extremely high heat input welding procedures, causing a decrease in both the yield strength and tensile strength, as well as the notch toughness of the weld deposit. Excessively low heat input may result in high-strength weld metal and may also decrease the notch toughness of the weld deposit. Optimum mechanical properties are generally obtained from both the weld metal and the HAZ if the heat input is maintained in the 30- to 80-kJ/in range.

Post-Northridge evaluation of fractured connections has revealed that excessively high heat input welding procedures were commonly used, confirmed by the presence of very large weld beads that, in some cases, exceeded the maximum limits prescribed by the D1.1 code. These may have had some corollary effects on weld metal and HAZ properties.

Material properties and connection design. While a welded structure acts as a one-piece unit, the material properties are not isotropic throughout all zones. When high demands are placed upon connections, each series of material properties must behave as expected.

One approach to obtaining acceptable connections is rigorous control of each of the material properties. Evidence exists that this can be done, even under very demanding conditions that simulate earthquake loading. There is an acceptable approach, however, that relies less on rigorous control of the mechanical properties, and compensates for these concerns by geometrically changing the connection. This alternative is discussed in "Cover-Plated Designs" section.

3.9.6 Workmanship

For severely loaded connections, good workmanship is a key contributor to acceptable performance. In welded construction, the performance of the structural system is often dependent on the ability of skilled welders to deposit sound weld metal. As the level of loading increases, dependence upon high-quality fabrication increases.

The role of codes. Design and fabrication specifications such as the *AISC Manual of Steel Construction* and the *AWS D1.1 Structural Welding Code—Steel* are typical vehicles that communicate minimum acceptable practices. It is impossible for any code to be so inclusive as to cover every situation that will ever be contemplated. These codes and specifications address minimum acceptable standards, relying upon the engineer to specify any additional requirements that supersede these minimum levels.

The D1.1 code does not specifically address seismic issues but does establish a minimum level of quality that must be achieved in seismic applications. Additional requirements are probably warranted. These would include requirements for nondestructive testing and notch tough weld deposits, and additional requirements for in-process verification inspection.

Purpose of WPS. Within the welding industry, the term *welding procedure specification,* or simply *WPS,* is used to signify the combination of variables that are to be used to make a particular weld. The WPS is somewhat analogous to a cook's recipe. It outlines the steps required to make a good-quality weld under specific conditions. It is the primary method to ensure the use of welding variables essential to weld quality. Also, it permits inspectors and supervisors to verify that the actual welding is performed in conformance with the constraints of the WPS.

Prior to the start of welding on a project, WPSs typically are submitted to the engineer for review. Many engineers with limited understanding of welding will delegate this responsibility to other individuals, often the owner's supplied inspection firm, to verify the suitability of the particular parameters involved. For critical projects, the services of welding engineers may be appropriate. Most importantly, WPSs are not simply pieces of documentation to be filed away—they are intended to be communication tools for maintenance of weld quality. It is essential that all parties involved with the fabrication sequence have access to these documents to ensure conformance to their requirements.

Effect of welding variables. A variety of welding variables determine the quality of the deposited weld metal. These variables are a function of the particular welding process being used, but the general trends outlined below are applicable to all welding processes.

Amperage is a measure of the amount of current flowing through the electrode and the work. It is a primary variable in determining heat input. An increase in amperage generally means higher deposition rates, deeper penetration, and more melting of base metal. The role of amperage is best understood in the context of heat input and current density, which are described below.

Arc voltage is directly related to arc length. As the voltage increases, the arc length increases. Excessively high voltages may lead to weld metal porosity, while extremely low voltages will result in poor weld bead shapes. In an electric circuit, the voltage is not constant but is composed of a series of voltage drops. For this reason, it is important to monitor voltage near the arc.

Travel speed is the rate at which the electrode is moved relative to the joint. All other variables being equal, travel speed has an inverse effect on the size of weld beads. Travel speed is a key variable used in determining heat input.

Polarity is a definition of the direction of current flow. Positive polarity (or reverse) is achieved when the electrode lead is connected lo the positive terminal of the dc power supply. The work lead would be connected to the negative terminal. Negative polarity (or straight) occurs when the electrode is connected to the negative terminal. For most welding processes, the required electrode polarity is a function of the design of the electrode. For submerged arc welding, either polarity could be utilized.

Heat input is generally expressed by the equation

$$H = \frac{60\,EI}{1000S}$$

where E represents voltage, I is current, and S is the travel speed in inches per minute. The resultant computation is measured in kilojoules per inch. The heat input of welding is also directly related to the cross-sectional area of the weld bead. High heat input welding is automatically associated with the deposition of large weld passes. The AWS D1.1 code does not specify heat input limits but does require that weld bead shapes meet prescribed requirements of maximum height and widths. This has an indirect effect of limiting heat input.

Current density is determined by dividing the welding amperage by the cross-sectional area of the electrode. The current density is therefore proportional to I/d^2. As the current density increases, there will be an increase in deposition rates as well as penetration.

Preheat and interpass temperatures are used to control cracking tendencies, typically in the base material. Excessively high preheat and interpass temperatures will reduce the yield and tensile strength of the weld metal as well as the toughness. When base metals receive little or no preheat, the resultant rapid cooling can promote cracking as well as excessively high yield and tensile properties in the weld metal, and a corresponding reduction in toughness and elongation.

All of the preceding variables are defined and controlled by the welding procedure specification. Conformance to these requirements is particularly sensitive for critical fabrication such as seismically loaded structures, because of the high demand placed upon welded connections under these situations.

Fit-up. *Fit-up* is the term that defines the orientation of the various pieces prior to welding. The AWS D1.1 code has specific tolerances that are applied to the as-fit dimensions of a joint prior to welding. It is critical that there is ample access to the root of the joint to ensure good, uniform fusion between the members being joined. Excessively small root openings or included angles in groove welds do not permit uniform fusion. Excessively large root openings or included angles result in the need for greater volumes of weld metal, with their corresponding increases in shrinkage stresses. This in turn increases distortion and cracking tendencies. The D1.1 tolerances for fit-up are generally measured in $^1/_{16}$-in increments. As compared to the overall project, this is a very tight dimension. Nevertheless, as it affects the root opening condition, it is critical in order to avoid lack of fusion, slag inclusions, and other unacceptable root conditions.

Field versus shop welding. Many individuals believe that the highest-quality welding naturally is obtained under shop welding conditions. While some aspects of field welding are more demanding than shop welding situations, the greatest differences are not technical but rather are related to control. For shop fabrication, the work force is generally more stable. Supervision practices and approaches are well understood. Communication with the various parties involved is generally more efficient. Under field welding conditions, maintaining and controlling a project seems to be more difficult. There are environmental challenges to field conditions, including temperature, wind, and moisture. However, those issues seem to pose less of a problem than do the management-oriented issues.

Generally, gasless welding processes, such as self-shielded flux cored welding, and shielded metal arc welding are preferred for field welding. Gas metal arc, gas tungsten arc, and gas-shielded flux cored arc welding are all limited due to their sensitivity to wind-related gas disturbances. Regarding managerial issues, it is imperative that field welding conditions receive an appropriate increase in the monitoring and control area to ensure consistent quality. The AWS D1.1 code imposes the same requirements on field welding as on shop welding. This includes qualification of welders, the use of welding procedures, and the resultant quality requirements.

3.9.7 Inspection

To ensure weld quality, a variety of inspection activities is employed. The AWS D1.1 code requires that all welds be inspected, specifically by means of visual inspection. In addition, at the engineer's discretion and as identified in contract documents, nondestructive testing may be required far finished weldments. This enables the engineer with a knowledge of the complexity of the project to specify additional inspection methodologies commensurate with the degree of confidence required for a particular project. For seismically loaded structures, and connections subject to high stress levels, the need for inspection increases.

In-process visual inspection. The D1.1 code mandates the use of in-process visual inspection. This activity encompasses those operations performed before, during, and after welding that are used to ensure weld quality. Before start-up, the inspector reviews welder qualification records, welding procedure specifications, and the contract documents to confirm that applicable requirements are met. Before welding is performed, the inspector verifies fit-up and joint cleanliness, examines the welding equipment to ensure it is in proper working order, verifies that the materials involved meet the various requirements, and confirms that the required levels of preheat have been properly applied. During welding, the inspector confirms that appropriate WPS parameters are being achieved and that the intermediate weld passes meet the various requirements. After welding is completed, final bead shapes and welding integrity can be visually confirmed. In spite of its apparent simplicity, effective visual inspection is a critical component for ensuring consistent weld quality.

Nondestructive testing. There are four major nondestructive testing methods that may be employed to verify weld integrity after the welding operations are completed. None is a substitute for effective visual inspection as outlined previously. No process is completely capable of detecting all discontinuities in a weld. The advantages and limitations of each method must be clearly understood in order to ensure an appropriate level of confidence is placed in the results obtained.

Dye penetrant inspection (PT) involves the application of a liquid which is drawn into a surface-breaking discontinuity, such as a crack or porosity, by capillary action. When the excess residual dye is removed from the surface, a developer is applied which will absorb the penetrant that is contained within the discontinuity. The result is a stain in the developer that shows that a discontinuity is present. Dye penetrant testing is limited to surface-breaking discontinuities. It has no ability to read subsurface discontinuities, but it is highly effective in accenting the discontinuities that may be overlooked, or may be too small to detect, by visual inspection.

Magnetic particle inspection (MT) utilizes the change in magnetic flux that occurs when a magnetic field is present in the vicinity of a discontinuity. The change in magnetic flux density will show up as a different pattern when magnetic dustlike particles are applied to the surface of the part. The process is highly effective in locating discontinuities that are surface-breaking or slightly subsurface. The magnetic field can be created in the material in one of two ways: the current is either directly passed through the material, or the magnetic field is induced through a coil on a yoke. The process is most sensitive to discontinuities that lie perpendicular to the magnetic flux path, so it is necessary to energize the part being inspected in two directions in order to fully inspect the component.

Ultrasonic inspection (UT) relies on the transmission of high-frequency sound waves through materials. Solid discontinuity-free materials will transmit the sound throughout the part in an uninterrupted manner. A receiver "hears" the sound reflected off of the back surface of the part being inspected. If a discontinuity is contained between the transmitter and the back of the part, an intermediate signal will be sent to the receiver, indicating the presence of a discontinuity. The pulses are read on a CRT screen. The magnitude of the signal received from the discontinuity indicates its size. The relationship of the signal with respect to the back wall is indicative of its location. UT is most sensitive to planar discontinuities, that is, cracks. UT effectiveness is dependent upon the operator's skill, so UT technician training and certification is essential. With the available technology today, UT is capable of reading a variety of discontinuities that would be acceptable for many applications, it is important that the acceptance criteria be clearly communicated to the inspection technicians so unnecessary repairs are avoided.

Radiographic inspection (RT) uses x-rays or gamma rays that are passed through the weld to expose a photographic film on the opposite side of the joint. X-rays are produced by high-voltage generators while gamma rays are produced by atomic disintegration of radioisotopes. Whenever radiographic inspection is employed, precautions must be taken to protect workers from exposure to excessive radiation. RT relies on the ability of the material to pass some of the radiation through, while absorbing part of this energy within the material. The absorption rate is a function of the material. As the different levels of radiation are passed through the material, portions of the film are exposed to a greater or lesser degree than the rest. When this film is developed, the resulting radiograph will bear the image of the cross section of the part. The radiograph is actually a negative. The darkest regions are those that were most exposed when the material being inspected absorbed the least amount of radiation. Porosity will be revealed as small dark round circles. Slag is generally dark and will look similar to porosity but will have irregular shapes. Cracks appear as dark lines. Excessive reinforcement will result in a light region.

Radiographic inspection is most effective for detecting volumetric discontinuities such as slag or porosity. When cracks are oriented perpendicular to the direction of a radiographic source, they may be missed with the RT method. Radiographic testing has the advantage of generating a permanent record for future reference. Effective interpretation of a radiograph and its implications requires appropriate training. Radiographic inspection is most appropriate for butt joints and is generally not appropriate for inspection of corner or T joints.

Applications for NDT methods. Visual inspection should be appropriately applied on every welding project. It is the most comprehensive method available to verify conformance with the wide variety of issues that can affect weld quality. In addition to visual inspection, nondestructive testing can be specified to verify the integrity of the deposited weld metal. The selection of the inspection method should reflect the probable discontinuities that would be encountered and the consequences of undetected discontinuities. Consideration must be made to the conditions under which the inspection would be performed such as field versus shop conditions. The nature of the joint detail (butt, T, corner, etc.) and the weld type (CJP, PJP, fillet weld) will determine the applicability of the inspection process in many situations. Magnetic particle inspection is generally preferred over dye penetrant inspection because of its relative simplicity. Cleanup is easy, and the sensitivity of the process is good. PT is normally reserved for applications where the material is nonmagnetic and MT would not be applicable. While MT is suitable for surface or slightly subsurface discontinuity detection only, it is in these areas that many welding defects can be located. It is very effective in crack detection and can be utilized to ensure complete crack removal before subsequent welding is performed on damaged structures.

Ultrasonic inspection has become the primary nondestructive testing method used for most building applications. It can be utilized to inspect butt, T, and corner joints, is relatively portable, and is free from the radiation concerns associated with RT inspection. UT is particularly sensitive to the identification of cracks, the most significant defect in a structural system. While it may not detect spherical or cylindrical voids such as porosity, the consequences of nondetection of these types of discontinuities are less significant.

3.9.8 Post-Northridge details

Prior to the Northridge earthquake, the special moment-resisting frame (SMRF) with the pre-Northridge beam-to-column detail was unchallenged with respect to its ability to perform as expected. This confidence existed in spite of a fairly significant failure rate that had been experienced when testing these connections in previous research. For purposes of this section, the pre-Northridge detail is considered to exhibit the following:

- CJP groove welds of the beam flanges to the column face, with weld backing left in place and with weld tabs left in place
- No specific requirement for minimum notch toughness properties in the weld deposit
- A bolted web connection with or without supplemental fillet welds of the shear tab to the beam web
- Standard ASTM A36 steel for the beam, and ASTM 572 grade 50 for the column, for example, no specific limits on yield strength or the F_yF_u ratio

As a result of the Northridge earthquake and research performed immediately thereafter, confidence in this detail has been severely shaken. Whether this detail, or a variation thereof, will be suitable for use in the future is unknown at the time of writing. More research must be performed, but we can speculate that, with the possible exception of small-sized members, some modification will be required in order to gain the expected performance from structural systems utilizing this detail.

As was previously stated, testing of this configuration had a fairly high failure rate in pre-Northridge tests. Still, many successful results were obtained. Further research will determine which variables are the most significant in predicting performance success. Some changes, however, have taken place in materials and design practice that should be considered. In recent years, recycling of steel has become a more predominant method of manufacture. Not only is this environmentally responsible, it is economical. However, in the process, residual alloys can accumulate in the scrap charge, inadvertently increasing steel strength levels. In the past 20 years, for example, the average yield strength of ASTM A36 steel has increased approximately 15%. Testing done with lower-yield-strength steel would be expected to exhibit different behavior than test specimens made of today's higher-strength steels (in spite of the same ASTM designation).

For practical reasons, laboratory specimens tend to be small in size. Success in small-sized specimens was extrapolated to apply to very large connection assemblies in actual structures. The design philosophy that led to a reduction in the number of special moment-resisting frames throughout a structure necessitated that each of the remaining frames be larger in size. This corresponded to heavier and deeper beams, and much heavier columns, with an increase in the size of the weld between the two rolled members. The effect of size on restraint and triaxial stresses was not evaluated in the laboratory, resulting in some new discoveries about the behavior of the large-sized assemblages during the Northridge earthquake.

There is general agreement throughout the engineering community that the pre-Northridge connection (as defined above) is no longer adequate and some modification will be required. Any deviation from the previous definition constitutes a modification for the purposes of this discussion.

Minor modifications to the SMRF connection. With the benefit of hindsight, several aspects of the pre-Northridge connection detail seem to be obviously deficient. Weld backing left in place in a connection subject to both positive and negative moments where the root of the flange weld can be put into tension is an obvious prescription for high-stress concentrations that may result in cracking. Failure to specify minimum toughness levels for weld metal for heavily loaded connections is another deficiency. The superior performance of the all-welded web versus the bolted web in past testing draws into question the assumption of load sharing between welds and bolts. Tighter control of the strength properties of the beam steel and the relationship to the column also seem to be obvious requirements.

The amount of testing that controls each of these variables has been limited to date. Some preliminary results suggest that tightly controlling all these variables will result in acceptable performance. At the time of writing, however, the authors know of no test of unmodified beam-to-column connections where the connection zone has remained crack-free when acceptable rotation limits were achieved. It is speculated that for smaller-sized members, this approach may be technically possible, although the degree of control necessary on both the material properties and the welding operations may not be practical.

Cover-plated designs. This concept uses short cover plates that are added to the top and bottom flanges of the beam. Fillet welds transfer the cover-plate forces to the beam flanges. The bottom flange cover plate is shop-welded to the column flange, and the bottom beam flange is field-welded to the column flange and to the cover plate. Both the top flange and the top flange cover plate are field-welded to the column flange with a common weld. The web connection may be welded or high-strength bolted. Limited testing of these connections has been done, with generally favorable results.

The cover-plate approach has received significant attention after Northridge because it offered early promise of a viable solution. Other methods may emerge as superior as time progresses. While the cover-plate solution treats the beam the same as other approaches (that is, it moves the plastic hinge into a region where ductility can be demonstrated), it concentrates all the loading to the column into a relatively short distance. Other alternatives may treat the column in a gentler manner.

Flange rib connections. This concept utilizes one or two vertical ribs attached between the beam flanges and column face. In a flange rib connection, the intent of the rib plates is to reduce the demand on the weld at the column flange and to shift the plastic hinge from the column face. In limited testing, flange rib connections have demonstrated acceptable levels of plastic rotation provided that the girder flange welding is correctly done.

Vertical ribs appear to function very similarly to the cover-plated designs but offer the additional advantage of spreading that load over a greater portion of that column. The single rib designs appear to be superior to the twin rib approaches because the stiffening device is in alignment with the column web (for I-shaped columns) and facilitates easy access to either side of the device for welding. It is doubtful that the single rib would be appropriate for box column applications.

Top and bottom haunch connections. In this configuration, haunches are placed on both the top and bottom flanges. In two tests of the top and bottom haunch connection, it has exhibited extremely ductile behavior, achieving plastic rotations as great as 0.07 rad. Tests of single, haunched beam-column connections have not been as conclusive; further tests of such configurations are planned.

Haunches appear to be the most straightforward approach to obtaining the desired behavior out of the connection, albeit at a fairly significant cost. The treatment to the column is particularly desirable, greatly increasing the portion of the column participating in the transfer of moment. Significant experience was gained utilizing the haunches for the repairs of the SAC-sponsored tests.

Reduced beam section connections. In this configuration, the cross section of the beam is intentionally reduced within a segment to produce a deliberate plastic hinge within the beam span, away from the column face. One variant of this approach produces the so-called dog-bone profile.

Reduced section details offer the prospect of a low-cost connection and increased performance out of detailing that is very similar to the pre-Northridge connection. Control of material properties of the beam will still be a major variable if this detail is used. Lateral bracing will probably be required in the area of the reduced section to prevent buckling, particularly at the bottom flange when loaded in compression.

Partially restrained connections. Several engineers and researchers have suggested that partially restrained connection details will offer a performance advantage over the special moment-resisting frames. The relative merits of a partially restrained frame versus a rigid frame are beyond the scope of this chapter. However, many engineers immediately think of bolted PR connections when it is possible to utilize welded connections for PR performance as well.

Illustrated in Fig. 3.37 are a variety of details that can be employed utilizing the PR concept. Detailing rules must be developed, and testing

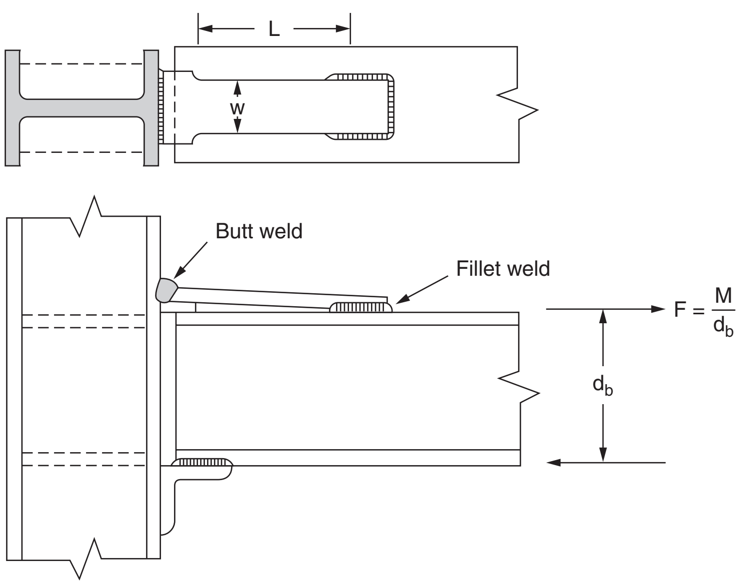


Figure 3.37 Examples of partially restrained connection details. (*Courtesy of The Lincoln Electric Company.*)

should be performed, before these details are employed. They are supplied to offer welded alternatives to bolted PR connection concepts.

3.10 References

1. American Institute of Steel Construction, *Manual of Steel Construction,* 2d ed., vol. I, LRFD, Chicago, IL, 1994.
2. American Welding Society, *Structural Welding Code—Steel,* D1.1, Miami, FL, 1998.

Chapter

4

Partially Restrained Connections

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4.1 Introduction

The American Institute of Steel Construction (AISC) specification has recognized semirigid (Type 3) or partially restrained (PR) construction since the 1940s (AISC, 1947). Because the design of Type 3/PR connections is predicated on a set of forces obtained from an advanced structural analysis that includes the connection deformation characteristic and because few if any design texts address this issue, this chapter will begin with an introductory discussion of PR connection and its effect on frame behavior. Once these issues are understood,

(*Courtesy of The Steel Institute of New York.*)

the connection design can proceed as for any other steel connection. For more detailed discussions of modeling and analysis issues for PR frames, the reader is referred to several excellent recent publications (Chan and Chui, 2000; Chen, 2000, Faella et al., 2000).

After the discussions on PR-frame design, examples for several types of PR connections, including T-stubs and flange-plate connections, are presented. The design of these connections for wind loads is straightforward, as this is only a matter of strength, and Examples 4.1 and 4.3 cover this case. Design for seismic loads is more complex, as both the ductility and energy dissipation of the connection needs to be considered. A large amount of research on PR bolted connections has been carried out after the 1994 Northridge earthquake, leading to the development of detailed design procedures for the use of these connections in areas of high seismicity (FEMA, 1997*b*, AISC 358, 2005). When properly designed, these connections exhibit excellent ductility and energy-dissipation capacity, distributing the deformation between ductile mechanisms in both the beam and the connection (Fig. 4.1). The seismic design examples presented in this chapter have been updated to reflect the proposed procedures in AISC 358 (2005). In its next edition, AISC 358 will deal with the connections shown in Examples 4.2 and 4.4.

Finally, it should be noted that the examples shown deal with connection behavior without explicitly treating the effect of the floor

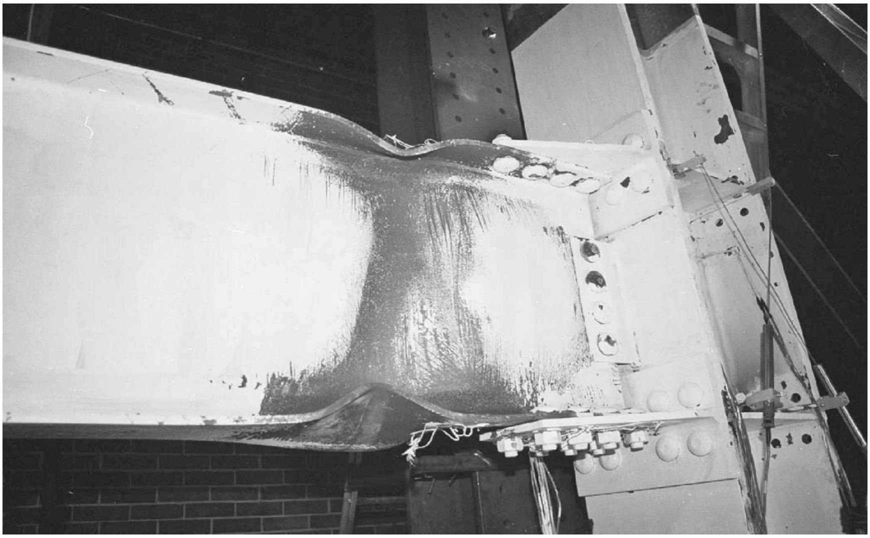

Figure 4.1 Cyclic performance of T-stub connection.

diaphragm. PR connections may benefit significantly from using reinforcing bars in the floor slabs to carry negative moments over the supports and to redistribute forces in the connection region. The design of this type of composite PR connection has been covered in detail in several publications (Leon et al. 1996, Leon, 1997), and a short summary of the topic is given at the end of the chapter.

4.2 Connection Classification

From the fifth to the eighth edition of the allowable stress specification (AISC, 1947, 1978), PR connections were categorized as Type 3 construction. The Type 3 design was predicated on the assumption that "connections of beams and girders possess a dependable and known moment capacity intermediate in degree between the rigidity of Type 1 (rigid) and the flexibility of Type 2 (simple)." This definition is confusing since it mixes strength and stiffness concepts, and was generally interpreted as referring to the initial stiffness (K_i) of the connection as characterized by the slope of its moment-rotation curve (Fig. 4.2*c*). Moreover, these specifications allowed the use of PR connections in "wind frames" under the Type 2 (simple framing) classification, where the connections were assumed as *simple* for gravity loads and *rigid* for lateral loads. Until the early 1980s, many steel frames were designed using PR connections through this artifice, which has disappeared from the most recent specifications.

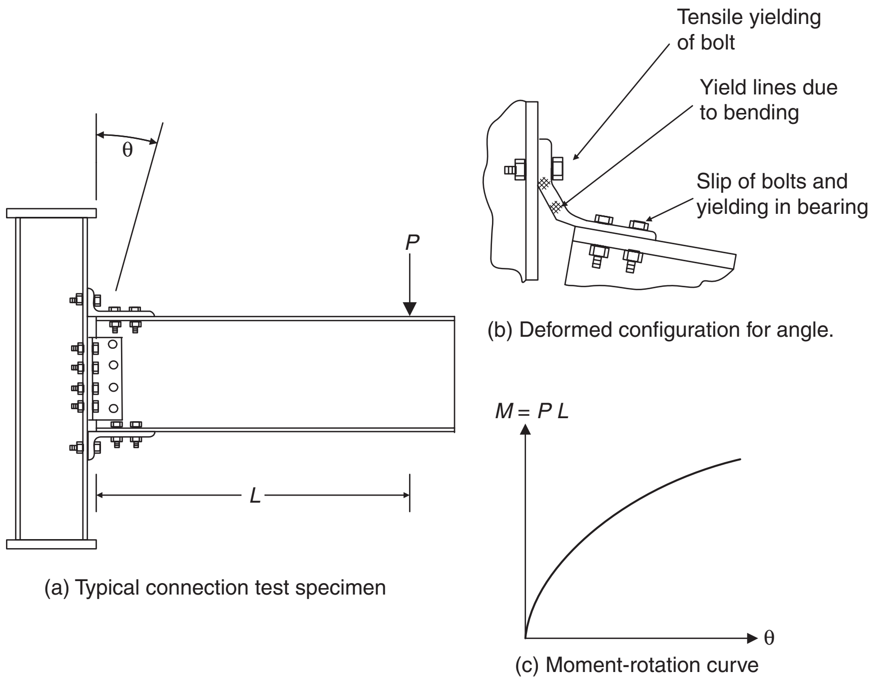

Figure 4.2 Derivation of M-θ curves from experiments.

Moment-rotation (M-θ) curves are generally assumed to be the best characterization of connection behavior for design purposes. These M-θ curves are generally derived from experiments on cantilever-type specimens (Fig. 4.2*a*). The moments are calculated directly from the statics of the specimen, while the rotations are measured over a distance typically equal to the beam depth. The rotation reported thus includes most deformation components occurring in the joint region. For the case of a top-and-seat angle shown in Fig. 4.2, these components include, among others, the elastic deformations due to the pullout of the angle, the rotation due to yield line formation in the leg bolted to the column due to bending, yielding of the angle leg attached to the beam in tension, slip of the bolts, and hole elongation due to bearing (Fig. 4.2*b*).

Because well-documented M-θ curves were rarely available in the open literature, because the design specifications provided no guidelines on how to implement this concept in practice, and because most commercial structural-analysis software could not handle nonlinear rotational springs, Type 3/PR construction has seldom been explicitly used until recently. As noted earlier, extensive use of PR connections was made through the artifice of Type 2 "wind" construction. While extensive research (Ackroyd and Gerstle, 1982) has shown this procedure to be generally safe, the final forces and deformations computed from a simplified analysis can be different from those using and advanced analysis program that incorporates the entire nonlinear M-θ relationship shown in Fig. 4.2*c*.

The description of Type 3 construction used in previous versions of the steel specification cannot properly account for the effect of connection flexibility at the serviceability, ultimate strength, or stability limit states. The first LRFD specification (AISC, 1986) recognized these limitations and changed the types of construction to fully restrained (FR) and partially restrained (PR) to more realistically recognize the effects of the connection flexibility on frame performance. The definition of PR connections in the first two LRFD versions of the specification (AISC 1986, 1993), however, conformed to that used for Type 3 in previous ASD versions. Research on PR connection behavior has led to more comprehensive proposals for connection classification (Gerstle, 1985, Nethercot, 1985, Bjorhovde et al., 1990, Eurocode 3 1992, to name but a few of the earlier ones) that clarify the combined importance of stiffness, strength, and ductility in connection design. The commentary of the more recent editions of the LRFD and unified specification (AISC 360, 2005) contain much more detailed discussion on connection classification schemes. The discussion here, which remains consistent with that in the previous edition of this book, is in substantial agreement with the main concepts that will appear in those commentaries.

4.2.1 Connection stiffness

As noted earlier, the connection stiffness can be taken as the slope of the M-θ curve. Since the curves are nonlinear from the start, it is possible to define this stiffness based on the tangent approach (such as for K_i in Fig. 4.3) or on a secant approach (such as K_{serv} or K_{ult}). A tangent approach is viable only if the analysis programs available can handle a continuous, nonlinear rotational spring. Even in this case, however, the computational overhead can be large and this option is recommended only for verification of the seismic performance of irregular structures. In most designs for regular frames, a secant approach will probably yield a reasonable solution at a fraction of the calculation effort required by the tangent approach. In this case, the analysis can be carried out in two steps by using linear springs. For service loads, a K_{serv} can be used for deflections and drift checks. The service secant stiffness can be taken at 0.0025 rad. A K_{ult}, based on a secant stiffness to a rotation of 0.02 rad, can be used for checks related to ultimate strength. Clearly, the deformations computed for the service load level will be fairly accurate, since the deviation of K_{serv} from the true curve

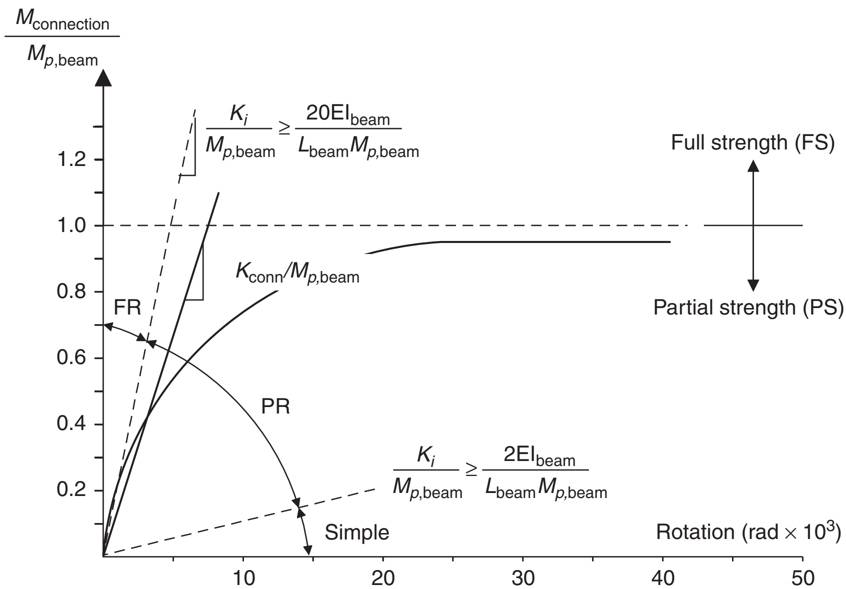

Figure 4.3 Connection classification by stiffness, strength, and ductility.

is typically small. On the other hand, the deformations computed for the ultimate strength case will probably not be very accurate, since there can be very large deviations and the linear spring K_{ult} can only be interpreted as an average. However, this approximation is probably sufficient for design purposes. Designers should be conscious that no theoretical proof exists that a secant stiffness such as K_{ult} will provide a conservative result.

The stiffness of the connection is meaningful only when compared to the stiffness of the connected members. For example, a connection can be classified as rigid (Type FR) if the ratio (α) of the connection secant stiffness at service level loads (K_{serv}) to the beam stiffness (EI/L), is greater than approximately 18 for unbraced frames. Generally connections with $\alpha < 2$ are regarded as pinned connections. Limits on the ranges of α cannot be established uniquely because they will vary depending on the limit state used to derive them. For regular frames, for example, one commonly used criterion to establish an upper limit is that the reduction in elastic buckling capacity due to the flexibility of the connections should not exceed 5% from that given by an analysis assuming rigid connections (Eurocode 3, 1992). Because this reduction in buckling capacity is tied to whether the frame is braced or unbraced, the value of 20 is suggested for unbraced frames, while a value of 8 is sufficient for braced frames. For continuous beams in braced frames, on the other hand, limits based on achieving certain percentage of the fixed-end moment or reaching a deflection limit seem more reasonable (Leon, 1994).

4.2.2 Connection strength

A connection can also be classified in terms of strength as either a full-strength (FS) connection or a partial-strength (PS) connection. An FS connection develops the full plastic moment capacity M_P of the beam framing into it, while a PS connection can only develop a portion of it. For classifying connections according to strength, it is common to nondimensionalize the vertical axis of the M-θ curve by the beam plastic moment capacity ($M_{p,\text{beam}}$) as is shown in Fig. 4.3. Connections not capable of transmitting at least 0.2 M_p at a rotation of 0.02 rad are considered to have no flexural strength. Because many PR connections do not exhibit a plateau in their strength even at large rotations, an arbitrary rotation value must be established to compare connection strength ($M_{p,\text{conn}}$) to the capacity of the beam. For this purpose a rotation of 0.02 rad is recommended by the author.

4.2.3 Connection ductility

Connection ductility is a key parameter either when the deformations are concentrated in the connection elements, as is the typical case in PR connections, or when large rotations are expected in the areas adjacent to the connections, as in the case of ductile moment frames with welded connections. The ductility required will depend on the flexibility of the connections and the particular application (that is, braced frame in a nonseismic area versus an unbraced frame in a high-seismic area).

A connection can be classified as ductile based on both its absolute and its relative rotation capacity (Fig. 4.4). The horizontal axes in Fig. 4.4 show both total connection rotations and connection ductilities. Three connection curves are shown: (a) two of the curvs are for connections in special moment frames (SMFs), one with hardening or non-degrading behavior (ND) and one with moderate degradation (D), and (b) one of the curves is for a degrading connections in an intermediate moment frame (IMF). The total rotation (in terms of milliradians or radians $\times 10^3$) is how typical moment-rotation curves for connection tests are reported. In general, only the envelopes of the cyclic results are shown, and a very coarse relative limit between ductile and nonductile connections can be set a total rotation of 0.04 rad.

The relative ductility index ($\mu = \theta_u/\theta_y$) can be used for comparing the rotation capacity of connections with similar moment-rotation characteristics. In order to compute a relative ductility (μ), a yield rotation (θ_y) must be defined. For PR connections, such as the one

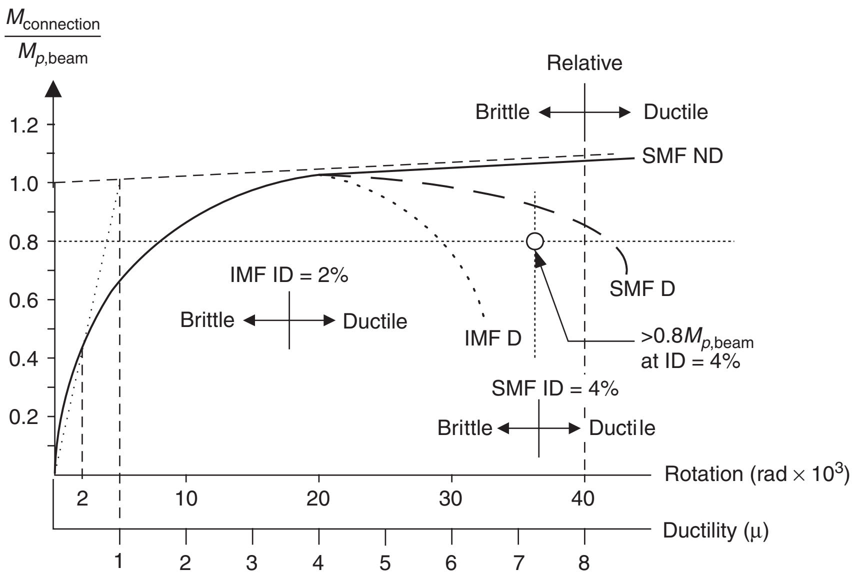

Figure 4.4 Possible connection ductility classification.

shown in Fig. 4.3, this definition is troublesome since a yield moment is difficult to determine. In this case, for the case of the connection in a special moment frame with no degradation (SMF, solid line) and for illustrative purposes only, the yield rotation is defined as the rotation at the intersection of the service and hardening stiffnesses of the connection. In general, relative ductilities of 3 or more are associated with well-detailed, nonseismic connections and general relative ductilities of 6 or more are associated with well-detailed, seismic connections.

Since the end of the work on the SAC (a join venture of research organizations in California) projects, the qualifications for connection performance has undergone two significant changes. First, the performance criteria for special and intermediate moment frames (0.04 rad of total connection rotation for SMF and 0.03 rad for IMF, both including an assumed 0.01 rad of elastic deformation) has been changed to the total interstory drifts (ID = 4 and 2%, for SMF and IMF respectively). These are also shown in Fig. 4.4, but their location in this figure is arbitrary with respect to the axes. In addition, the original requirement that the connection capacity at 0.04 rad should not decrease by more than 20% from its maximum has been changed to a requirement that at 4% drift SMF connections should not have less than 80% of the nominal flexural capacity of the beam. These two are significant changes, as direct conversions between both interstory drift and connection rotation, and connection and beam strength are not possible.

Both the absolute and relative rotation capacities need to take into account any strength degradation that may occur as a result of local buckling or slip, particularly under cyclic loads. The behavior of the connections shown by the solid (SMF ND), dashed (SMF D), and dotted (IMF D) lines in Fig. 4.4 can lead to significant differences in frame behavior, especially with respect to strength and stability. Finally, it should be emphasized that the limits discussed above, with the exception of the interstory drift and $0.8M_{p,\,\text{beam}}$ ones appearing in ASIC 360 (2005), are based purely on the opinions of the author.

Limits for ductility criteria, such as those described previously, are only now being developed, but this issue is highlighted here to remind designers that analysis assumptions (unlimited rotational ductility, in general) must be consistent with the detailing of the connection. This is true for both PR and FR frames.

4.2.4 Derivation of M-θ curves

As noted earlier, M-θ curves have typically been derived from experiments. Many of these tests have been collected into databases (Ang and Morris 1984, Goverdhan, 1984, Nethercot, 1985, Kishi and Chen, 1986, Chan and Chiu, 2000, for example). Based on these databases, equations for the complete M-θ curves for different types of connections have been proposed. However, numerous important variables, such as the actual yield strength of the materials and the torque in the bolts, are generally poorly documented or missing for many of these tests. Thus many of the M-θ curves and equations available from these databases cannot be considered as reliable. In addition, care should be exercised when utilizing tabulated moment-rotation curves not to extrapolate to sizes or conditions beyond those used to develop the database since other failure modes may control (ASCE, 1997).

Two approaches have recently become practical alternatives and/or complements to experimental testing in developing M-θ curves. The first alternative is a detailed, nonlinear finite-element analysis of the connection. While time-consuming because of the extensive parametric studies required to derive reliable M-θ curves, this approach has gone from a pure research tool to an advanced design office tool in just a few years thanks to the tremendous gains in computational power available in new desktop workstations.

The other approach is the one proposed by the Eurocodes and commonly labeled the "component approach." In this case each deformation mechanism in a joint is identified and individually quantified through a series of small component tests. These tests are carefully designed to measure one deformation component at the time. Each of these components is then represented by a spring with either linear or

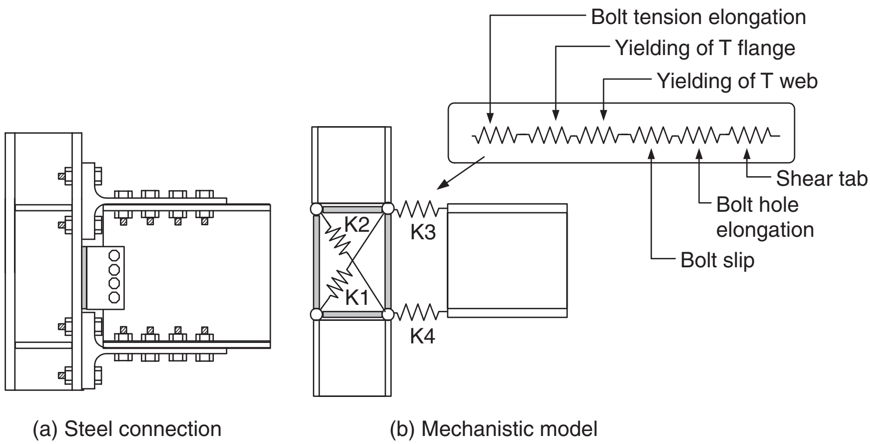

Figure 4.5 Component model for a T-stub connection.

nonlinear characteristics. These springs are arranged in series or in parallel and the overall M-θ curve derived with the aid of simple computer programs that conduct the analysis of the spring system. Figure 4.5 shows a typical model for a T-stub connection. In this example the $K1$ and $K2$ springs model the panel zone deformation due to shear, while springs $K3$ and $K4$ model the bending deformations of the T stubs. Springs $K3$ and $K4$ are made up from the contributions of several other springs that model different deformation components (Fig. 4.5*b*).

4.2.5 Analysis

For many types of connections, the stiffness at the service load level falls somewhere in between the fully restrained and simple limits, and thus designers need to account for the PR behavior. The M-θ characteristic can be obtained from experiments or models as described in the previous section. The effect of PR connections on both, force distribution and deformations in simple systems, will be illustrated with two short examples.

Figure 4.6 shows the moments and deflections in a beam subjected to a uniformly distributed load. The horizontal axis is logarithmic and shows the ratio of the connection to beam stiffness ($\alpha = K_{serv}L/EI$). The deformations rage from that of a simply supported beam ($\Delta = 5wL^4/384EI$) for a very flexible connection ($\alpha \rightarrow 0$) to that of a fixed beam ($\Delta = wL^4/384EI$) for a very stiff connection ($\alpha \rightarrow \infty$). From both the deflection and force-distribution standpoints, for a range of $15 < \alpha < \infty$ the behavior of the connection is essentially that of a fixed beam.

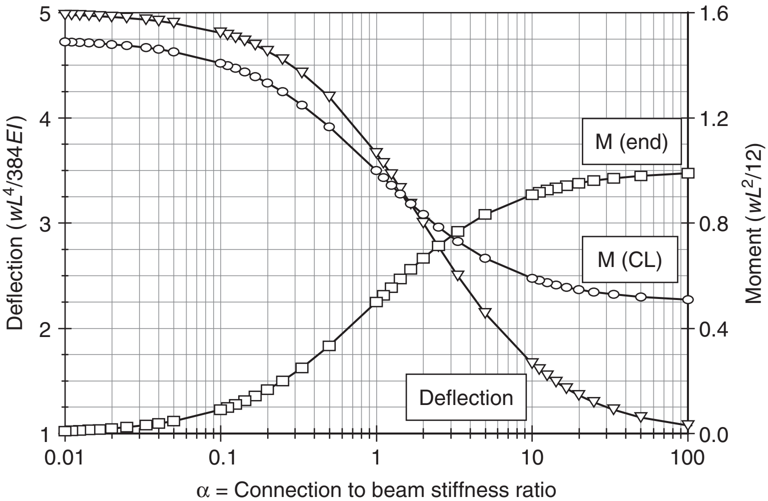

Figure 4.6 Moments and deflections for a beam under a uniformly distributed load with PR connections at its ends.

Similarly, for a range of $0 < \alpha < 0.3$, the beam is essentially simply supported. Note that the ranges given here were selected arbitrarily, and that they will vary somewhat with the loading condition. This is why, as was noted earlier in the discussion of connection stiffness, the selection of limits for α to separate FR, PR, and simple behavior are not straightforward. It is important to note, however, that the horizontal axis of Fig. 4.6 is logarithmic. This means that apparently large changes in connection stiffness actually result in much smaller changes in forces or deformations. This lack of sensitivity is actually what allows us to design PR connections by simplified methods, since it means that the connection stiffness does not need to be known with great precision.

Figure 4.7 shows the results of an analysis for the general case of a one-story, one-bay frame with springs both at the connections to the beam (K_{conn}) and at the base of the structure (K_{base}). A simple formula for the drift cannot be written for this general case. Figure 4.7 shows the drifts for five levels of base fixity ($\alpha_{base} = K_{base} H_e/EI_{column} = 0$, 1, 2.5, 5, 10, and ∞) versus a varying $\alpha_{beam} = (K_{conn}\ L/EI)$. The calculations are for a frame with an $I_{beam} = 2000$ in^4, $L = 288$ in, $I_{column} = 500$ in^4, $H = 144$ in, a concentrated horizontal load at the top of $P = 2.4$ kips, and a distributed load on the beam of $w = 0.08333$ kip/in. The vertical axis gives the deflection as a multiplier (τ) of the fully rigid case, where $K_{conn} = K_{base} = \infty$. The drift value for the latter is 0.025 in. For the case of $K_{base} = \infty$, as the connection stiffness decreases, the deflection reduces to that of a cantilever subjected to $P/2$ (τ =

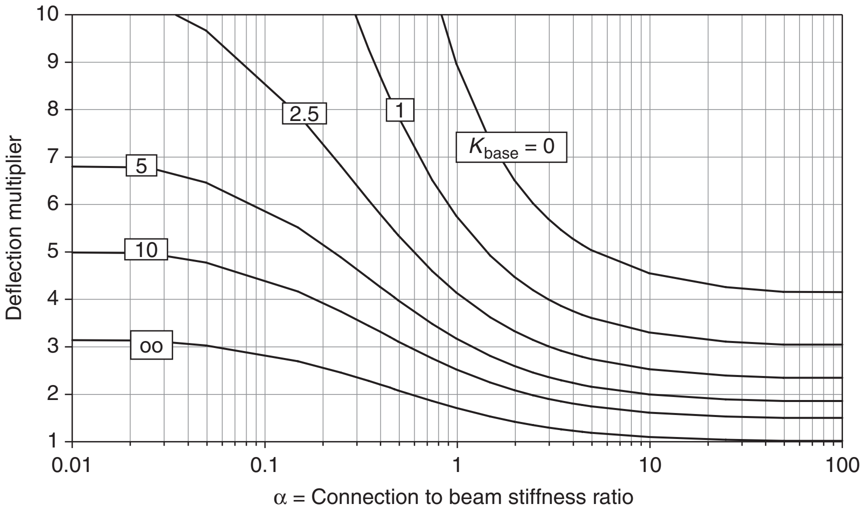

Figure 4.7 Drifts of a simple frame with various degrees of base fixity and connection stiffness.

3.25). For the other extreme ($K_{base} = 0$), the deflections increase rapidly from $\tau = 4.06$ as the stiffness of the connection is decreased since we are approaching the unstable case of a frame with pins at all connections as $\alpha \rightarrow 0$. Figures such as this indicate the wide range of behavior that PR connections can provide, and the ability of the designer to use the connection stiffness to tailor the behavior of the structure to its performance requirements.

Another very important lesson to be drawn from Fig. 4.7 is the large effect of the base fixity on frame drift. While it is common to assume in the analysis that the column bases are fixed, such degree of fixity is difficult to achieve in practice even if the column is embedded into a large concrete footing. Most footings are not perfectly rigid or pinned, with the practical range probably being $1 < K_{base} < 10$. As can be seen in Fig. 4. 6, the difference in drift between the assumption of $K_{base} = \infty$ (perfect base fixity) and a realistic assumption ($K_{base} = 10$) ranges from approximately 50% when K_{conn} is ∞ to approximately 300% when α is 0.

Figure 4.7 indicates that there are infinite combinations of K_{base} and K_{conn} for a given deflection multiplier. Consider the case of a one-story, one-bay frame with the properties given for Fig. 4.6. For a target deflection multiplier of, say, 3, one can design the frame with a pinned base and a K_{conn} approaching infinity ($\alpha = 0$), or one can design a rigid footing with a connection having an $\alpha = 2$ (pinned). This flexibility in design is what makes PR-connection design both attractive and somewhat disconcerting. It is attractive because it provides the designer with a wide spectrum of possibilities in selecting the structural members and their connections. It is disconcerting because most designers do not have extensive experience with PR analysis and PR frame behavior.

There are currently numerous good texts that address the analysis and design of PR frames (Bjorhovde et al., 1988, 1992, 1996; Chen and Lui, 1991; CTBUH, 1993; Chen and Toma, 1995; Chen et al., 1995; Leon et al., 1996; Chen, 2000; Faella et al., 2000; Chan and Chui, 2000). There is a considerable range in the complexity of the analysis approaches proposed in the literature. The appropriate degree of sophistication of the analysis depends on the problem at hand. When incorporating connection restraint into the design, the designer should take into account the effect of reduced connection stiffness on the stability of the structure and the effect of connection deformations on the magnitude of second-order effects (ASCE, 1997). Usually design for PR construction requires separate analysis to determine the serviceability limit state and the ultimate limit state because of the nonlinear nature of the M-θ curves.

4.3 Design of Bolted PR Connections

The design of a connection must start from a careful assessment of its intended performance. This requires the designer to determine the performance criteria with respect to stiffness (FR, PR, or simple), strength (FS or PS), and ductility. The stiffness is critical with respect to serviceability, while strength and ductility are critical with respect to life-safety issues. These criteria must be consistent with the model assumed for analysis. From Fig. 4.7, if an assumption of a rigid connection was made in the analysis, the resulting connection will typically be fully welded, welded-bolted, or a stiffened thick end-plate type. Similarly, if the connection was assumed as simple, then a shear plate welded to the column and bolted to the beam or angles bolted to both column and beam are appropriate.

If explicit use of PR behavior was made in the analysis, in the form of a rotational spring with a given K_{serv}, then a wide variety of connections can be chosen, ranging from an end plate (close to FR/FS performance) to top-and-seat angles (close to simple performance). The key here is to match the K_{serv} of the connection as designed to that assumed in the analysis. The matching should be done at the service level because drift and deflection criteria will probably govern the design in modern steel frames. The stiffness of the connection should be checked with at least the component model approach (Fig. 4.5). Since the stiffness of the connection will be dependent on the actual configuration of the connecting elements and the size of the framing members, it is possible to adjust the stiffness to match that assumed in design.

The ultimate strength and ductility of the connection as designed must also be compatible with that assumed in design. In this case it is imperative to identify all possible failure modes for the connection as designed. Moreover, it is necessary to understand the hierarchy of failure modes so that modes are excluded. Table 4.1 (FEMA, 1997*a*) shows a proposed hierarchy for seismic design of a variety of connections: column-welded–beam-bolted (CW-BB), column-bolted–beam-bolted or T stub (CB-BB), end plates (EP), top-and-seat connections (TS). The table indicates the type of failure associated with each

TABLE 4.1 Failure Modes for Bolted Connections

Connection type	CW-BB	CB-BB	EP	TS
Strength (FS or PS)	FS	FS	FS/PS	PS
Stiffness (FR or PR)	FR	FR/PR	PR	PR
Ductile:				
Slippage of slip-critical (friction) bolts	1	1		
Flexural beam yielding adjacent to nodal zone	2	2	1	
Yielding of connecting elements in tension	3	3		2
Formation of yield lines in connecting elements		4	2	1
Yielding of slab reinforcement in tension				
Panel zone yielding	4	5	3	3
Limited local buckling	5	6	4	4
Semiductile:				
Elongation of bolt holes due to bearing	6	7		5
Yielding of bolts to column flange in tension	8	9	5	6
Shear yielding of bolts to beam flange	7	8		7
Severe local buckling of beam flange	9	10	6	8
Brittle:				
Fracture of welds between column and plate	A			
Fracture/failure of shear connection to web	A	A	A	A
Bearing/crushing failure of concrete				
Fracture of shear studs and rebar				
Fracture of beam flange due to local buckling	A	A	A	
Shear failure of bolts	A	A	A	A
Tensile failure of bolts (including prying action)		A	A	A
Fracture of beam through net section		A	A	A
Fracture of connecting element through net section		A	A	A
Column web failure (yielding, crippling, buckling)	A	A	A	A
Edge distance or spacing failure of bolts	A	A	A	A
Block shear	A	A	A	A

Note: "A" indicates a brittle failure mode that should be carefully checked in design; CW-BB = column-welded–beam-bolted connections; CB-BB = column-bolted–beam-bolted; EP = end plate; TS = top-and-seat angles with double web angles; PR-CC = partially restrained composite connection.

mechanism (ductile, semiductile, or brittle), and lists the ductile and semiductile mechanisms in descending order of desirability. This table is arbitrary and reflects the biases of the author. As an example of how this hierarchy can be achieved for a T stub, Fig. 4.8 shows the possible yielding [mechanisms (1) to (9) in likely

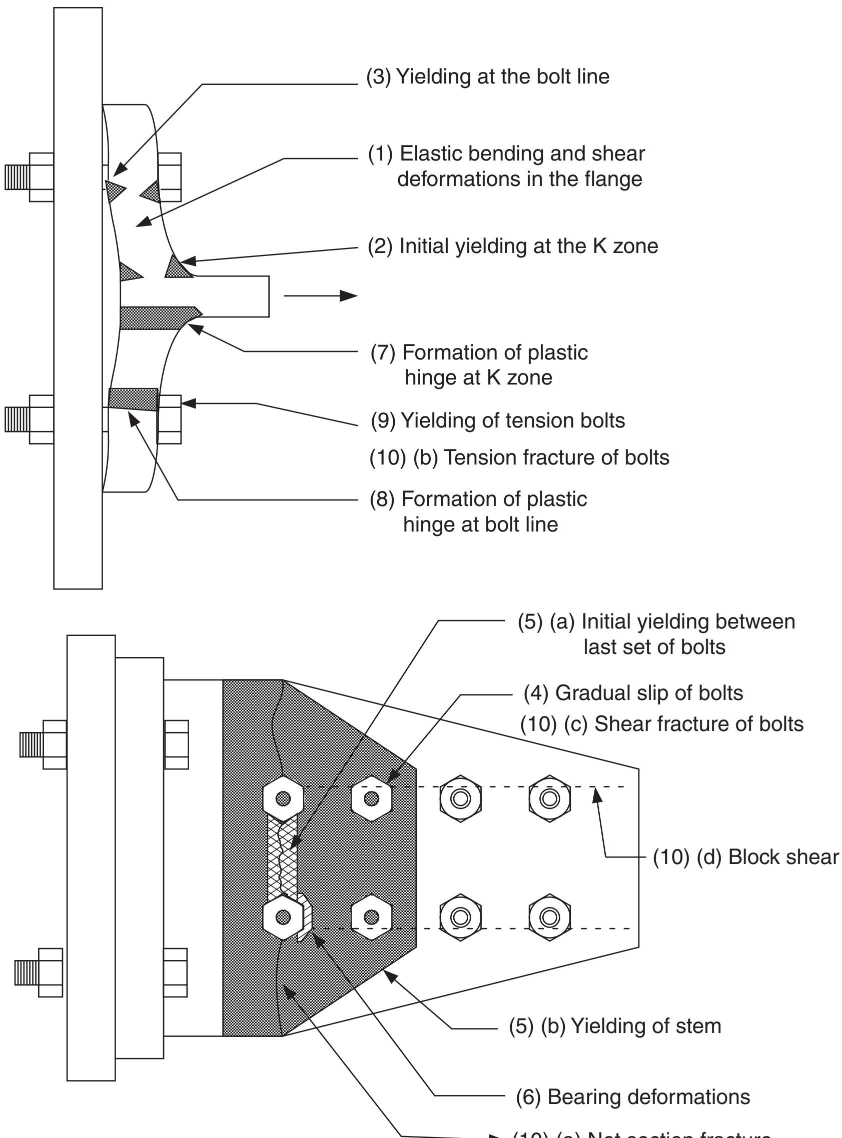

Figure 4.8 Yielding and fracture mechanisms in a T-stub connection.

order of occurrence] and fracture mechanisms [mechanism (10), any of which will led to connection failure]. For many PR connections the numerous sources of deformations provide considerable ductility but complicate the design. Designers are encouraged to develop their own lists and rankings based on their experience and regional preferences of fabricators and erectors.

Special note should be made of the fact that the material properties play an important role in connection performance. In particular, the separation between the expected yield ($R_y F_y$) and expected ultimate strength ($R_t F_u$) of the material is a key factor. As our understanding of the failures in steel frames during the 1994 Northridge earthquake improves, it is clear that material performance played an important role in some of the failures encountered. Issues related to the ductility and toughness of the base materials for both welds and bolts, installation procedures, QA/QC in the field, and need for new, tighter material specifications have received considerable attention (FEMA, 1997*a*). Designers should strive to obtain the latest information in this area so that future failures can be avoided.

The design process outlined places a heavy additional burden on designers both in terms of professional responsibility and continuing education, not to mention substantial additional design time. Two important points need to be made with respect to these issues. First, as our designs become more optimal with respect to both strength and stiffness, many of the traditional assumptions made in design need to be carefully reexamined. These include, for example, serviceability criteria based on substantially different partition and cladding systems than those used today. Second, these optimized systems are far more sensitive to the assumptions about connection behavior since typically far fewer moment-resisting connections are used in steel frames today than 20 years ago.

In this section the fundamentals of design for full-strength, fully restrained (FS/FR) bolted connections will be discussed first, followed by that for partial-strength, partially restrained (PS/PR) ones. The design for both seismic and nonseismic cases will be discussed. The emphasis will be on understanding the basic steps in connection design and developing an understanding of the crucial mechanisms governing their behavior.

4.3.1 Column-welded–beam-bolted connections

The design and behavior of column-welded–beam-bolted (CW-BB) connections (Fig. 4.9) has been discussed extensively by Astaneh-Asl (1995) and Schneider and Teeraparbwong (2002). The mechanistic model for this type of connection, labeled column-bolted–beam-bolted

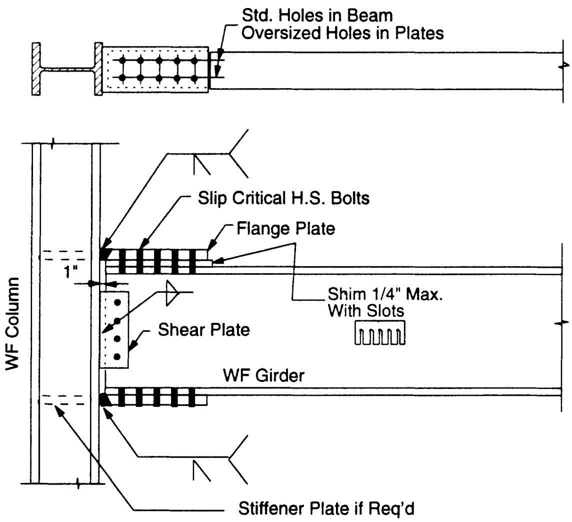

Figure 4.9 Typical CW-BB connection (Astaneh-Asl, 1995).

(CB-BB), is essentially the same as that shown in Fig. 4.5 for a T-stub connection. The main differences are that the springs representing the tension elongation of the bolts and the yielding in the flange have to be replaced by a spring that represents the behavior of the weld between the column flange and the beam flange.

Table 4.1 lists the main failure modes for this type of connection. In general the desired failure mechanisms will be slip of the bolts followed by yielding of the beam and the connection plate. The main failure modes to avoid are brittle failure of the welds, shear failure of the bolts, and a net section failure in the connecting plate or beam. With this hierarchy established, it is possible to develop a design strategy, as outlined in the steps shown below, for the design of these connections under monotonic loads.

The design of any connection subjected to seismic loads is similar in principle to the static design, except that a capacity-design approach must be followed. In this context, capacity design implies that the connection must be designed to behave in a ductile manner under the maximum expected forces that can be introduced by the framing members. Thus, for CW-BB connections, the welds need to be strong and tough enough such that the weld strength does not control and fracture problems related to the welding procedures and materials are eliminated. For CW-BB connections, yielding should be limited to the connection plate or the beam flange. This requires a careful assessment of the minimum and maximum capacities associated with each of the springs in Fig. 4.4, since the forces are inertial rather than gravity-type. For seismic design of connections, AISC 358-05 require that the expected (or mean) strength of the beam be used rather than its nominal (or 5% fractile) strength. To accomplish this, nominal yield and ultimate strength values are multiplied by either a R_y or R_t factor, which varies with the material type. In addition, to account for peak connection strength, strain hardening, local restraint, additional reinforcement, and other connection conditions and additional factor (C_{pr}) is used, where C_{pr} is taken as the average of the yield plus ultimate strength divided by the yield strength. C_{pr} need not be taken as greater than 1.2. This capacity-design approach is different from the static (that is, nonseismic) case where the connection can be designed for forces derived from the structural analysis, and without regard to what the actual ultimate capacity and failure mode of each of the connection components is.

Before looking at examples of CW-BB connections for both static and seismic loading cases, a number of important design issues need to be understood. These issues, discussed in detail below, are of particularly significance for CW-BB connections, but the principles involved are applicable to most strong PR connections:

1. *Proportioning of flange connection*: Whenever possible the yield strength of the connection elements (top and bottom plates) should be matched to that of the beam flange. This will ensure that distributed yielding takes place and that severe local buckling will not ensue. Severe local buckling can result in an early fracture of the beam flanges if cyclic loads are present. Astaneh-Asl (1995) recommends that for yielding on the gross section:

$$b_p\, t_p\, R_y\, F_{yp} \cong b_f\, t_f\, R_y\, F_{yf} \tag{4.1a}$$

where b and t are the width and thickness and the subscripts p and f refer to the plate and beam flange, respectively. Usually, the expected yield strength of the materials is not known when the design is done. For designs not involving seismic forces, the nominal material properties, as opposed to the nominal ones, can be used throughout. For the case of seismic forces the same assumptions can be made with regards to sizing the plate, but both the R_y and C_{pr} factors must be applied to avoid undesirable modes of failure. To avoid a tensile rupture of the flange, by AISC 358 (2005), Section F13:

$$d_b \le \frac{1}{2} b_f \left(1 - \frac{Y_t F_y}{F_u}\right) - \frac{1}{8} \tag{4.1b}$$

where $Y_t = 1$ if $(F_y/F_u) < 0.8$ or 1.1 otherwise. In order to ensure a ductile failure, the ratio of the effective area (A_e) to the gross area (A_g) of the plate should be at least:

$$\frac{A_e}{A_g} \geq \frac{R_y F_y}{R_g F_u} \tag{4.2}$$

2. For the case of seismic loads another key issue is the design of the welds to the column flange. In this area there are recent, detailed guidelines proposed by SAC (FEMA, 1995, 1997*a*) and AISC 358 (2005). The AISC provisions require that a welding procedure specification (WPS) be prepared as required by AWS D1.1 (AWS, 2005). AWS D1.1 provides detailed procedures for welding (see Chap. 3) and this standard should become familiar to all structural engineers. In addition, a minimum Charpy V-Notch test (CVN) toughness of 20 ft-lb at –20°F is required of all filler metal by the seismic AISC specification.

3. *Local buckling criteria*: The current limits suggested by AISC (2005) $0.38\sqrt{(E/F_y)}$ for b/t in beam flanges in compression and $3.76\sqrt{(E/F_y)}$ for webs in flexural compression seem to provide a reasonable limit to ensure that the nominal plastic moment capacity of the section is reached. For seismic applications, these limits have been tightened somewhat to $0.30\sqrt{(E/F_y)}$ for b/t in beam flanges and something less than $3.14\sqrt{(E/F_y)}$ for webs in flexural compression to ensure not only that the capacity can be reached, but also that sufficient rotational ductility is available. The typical buckle that forms when these criteria are met is a smooth, small local buckle. This precludes the development of a sharp buckle that may lead to fracture under reversed inelastic loading. The current limits on web slenderness also seem to provide reasonable limits although the actual performance will be tied to the detailing of the web connections and whether composite action is expected. The slenderness of the connection plates, measured between the weld to the column flange and the centerline of the first row of bolts, should also be kept as low as practicable to prevent the formation of a local or global buckle in this area. Current criteria for unsupported compression elements are applicable in this case.

4. *Bolts*: The bolt group should be designed not only to prevent a shear failure of the connectors but also to provide adequate performance during the slipping phase of the moment-rotation behavior. Since slip provides a good energy-dissipation mechanism, it is prudent to design the connection such that the slip occurs well above the

service load but also below the ultimate strength of the connection. To meet this criterion, Astaneh-Asl (1995) recommends that the nominal slip resistance (F_{slippage}) be such that

$$1.25F_{\text{service}} \le F_{\text{slippage}} \le 0.80F_{\text{ultimate}}$$

where F_{service} corresponds to the nominal slip strength of the bolt group and F_{ultimate} corresponds to the nominal shear strength of the bolts.

5. *Web connection design*: The design of the web connection is usually made without much regard to the contribution of this part of the connection to the flexural strength of the joint unless the flange connections carry less than 70% of the total moment (AISC, 1992). It is clear from the performance of MRF's during the Northridge earthquake that careful attention should be paid to ensure that the web connection is detailed to provide rotational ductility and strength that are compatible with the action of the flanges. Astaneh-Asl (1995) suggests that the shear plates be designed to develop the plastic moment strength of the web:

$$h_p\, t_p\, (0.6F_{yp}) > h_{gw}\, t_{gw}\, (0.6F_{yw})$$
$$h_p^2\, t_p\, F_{yp} > h_{gw}\, t_{gw}\, F_{yw}$$

where h and t are the depth and thickness, F_y is the yield strength, and the subscripts p and gw refer to the shear plate and the beam web, respectively. Here again, allowances should be made for the steel overstrength (say R_y = 1.1 to 15). Failure modes to be avoided include bolt shear, block shear, net area fractures, and weld fractures.

Design Example 4.1: Design a full-strength connection between a W 21 × 62 girder and a W 14 × 120 column. Both sections are A572 GR 50. Design for wind loads assuming the analysis shows a maximum moment (M_u) of 425 kip-ft (5100 kip-in) and a maximum shear (V_u) of 75 kips. The service moment (M_{serv}) is 140 kip-ft (1680 kip-in). Assume A325X 7/8-in-diameter bolts.

1. Check local buckling:
   Flange:

$$\left(\frac{b}{t}\right) = 6.1 \le 0.38\sqrt{\frac{E}{F_y}} = 9.2 \qquad \text{ok}$$

Web:

$$\left(\frac{h}{t_w}\right) = 46.9 \le 3.76\sqrt{\frac{E}{F_y}} = 90.6 \qquad \text{ok}$$

2. Check net area fracture versus gross section yielding of the girder flange by AISC (2005) Section F13. (Note: $Y_t = (F_y/F_u) = 50/65 = 0.76 < 0.8$ so $Y_t = 1.0$; for the W 21 × 62, $b_f = 8.24$ in, $t_f = 0.615$ in, $d = 20.99$ in, and $S_x = 127$ in^3; holes for 7/8-in bolts assumed as 1 in in diameter):

$$F_u A_{fn} = (65)(8.24 - 2)(0.615) = 249.4 \text{ kips}$$
$$Y_t F_y A_{fg} = (1.0)(50)(8.24)(0.615) = 253.4 \text{ kips}$$

Since the net section governs ($F_u A_{fn} < Y_t F_y A_{fg}$), the moment capacity is

$$M_n = \frac{F_u A_{fn}}{A_{fg}} S_x = \frac{(65)(8.24 - 2)(0.615)}{(8.24)(0.615)}(127) = 6251 \text{ kip-in}$$

$$\phi M_n = (0.9)(6251) = 5626 \text{ kip-in} > M_u = 5100 \text{ kip-in} \quad \text{ok}$$

$$F_{\text{flange}} = F_{\text{plate}} = \frac{\frac{5100}{0.9}\text{ kip-in}}{20.99 \text{ in}} = 270.0 \text{ kips}$$

3. Determine the size of the flange plate, assuming that the plate thickness (t_p) will be 5/8 in and that the plate is 50 ksi. Balancing the plastic capacity of the plate against that of the beam, gives a plate width (b_p) of

$$b_p \geq \frac{R_y F_{\text{beam}}}{t_p F_{yp}} = \frac{(1.1)(270.0)}{(0.625)(50)} = 9.50 \text{ in} \qquad \text{try } b_f = 9.5 \text{ in}$$

4. Check gross (A_{pg}) and net (A_{pn}) areas for the plate:
   Gross section:

$$\phi A_{pg} F_y = (0.9)(9.50 \times 0.625)(50) = 267.2 \text{ kips}$$

Net section:

$$\phi A_{pn} F_u = (0.75)((9.50 - 2) \times 0.625)(65)$$
$$= 228.5 \text{ kips} < F_{\text{plate}} \qquad \text{no good}$$

Increase plate thickness to 0.75 in (net section capacity = 274.2 kips > F_{plate}, ok)

5. Determine number of A325X 7/8-in bolts required for shear in the flanges:

$$N_{\text{bolts}} = \frac{267.3}{27.1} = 98 \rightarrow 10 \text{ bolts}$$

Assuming a gage of 5.0 in, this means the edge distance for a 9.5-in-wide plate are approximately equal to the minimum required. Assuming (a) a bolt spacing of $3d$, (b) a distance between the last bolt and the weld at the column flange of equal to 4 in, and (c) a distance of 2 in between the centerline of the last bolt and the end of the plate, the length is 16.5 in:

6. Check bolt bearing on the beam flange:

$$F_{\text{bearing}} = N_{\text{bolts}}(\varphi R_n) = (10)(0.75 \times 102) = 767 \text{ kips} \qquad \text{ok}$$

7. Check bolt service load slip capacity:

$$\begin{aligned} M_{\text{slip}} &= (10 \text{ bolts})(10.2 \text{ kip/bolt})(20.99 \text{ in}) \\ &= 2141 \text{ kip-in} > 1.25\,M = 2100 \text{ kip-in} \end{aligned}$$

8. Check block shear: Assume shear failure along the bolts and tensile failure across bolt gage and $U_{bs} = 1.0$:

$$A_{gv} = 2\left(16.5 - 4 - \frac{1}{2}\right)(0.75) = 18.00 \text{ in}^2$$

$$\begin{aligned} A_{gt} &= (5)(0.75) = 3.75 \text{ in}^2 \\ A_{nv} &= 18.00 - 2(3.5 \times 1)(0.75) = 12.75 \text{ in}^2 \\ A_{nt} &= 3.75 \times 2\,(1 \times 0.75) = 2.25 \text{ in}^2 \end{aligned}$$

$$\begin{aligned} R_n &= 0.6\,F_u\,A_{nv} + U_{bs}\,Fu\,A_{nt} \le 0.6\,F_y A_{gn} + U_{bs}\,F_u\,A_{nt} \\ &= 0.6(65)(12.75) + (1.0)(65)(2.25) \le 0.6(50)(18.00) \\ &\quad + (1.0)(65)(2.25) \\ &= 643.5 \text{ kips} \le 686.2.7 \text{ kips} \\ \phi R_n &= (0.75)643.5 \text{ kips} = 482.6 \text{ kips} \end{aligned}$$

9. Determine weld size: The weld thickness, based on a 70-ksi electrode is

$$\begin{aligned} t_{\text{weld}} &= \frac{F_{\text{plate}}}{0.6\,F_{\text{Exx}} \times b_p} \\ &= \frac{267.3 \text{ kips}}{0.6 \times 70 \text{ ksi} \times 9.5 \text{ in}} = 0.67 \text{ in,} \end{aligned}$$

say 3/4 in (use full-penetration weld)

10. Detail the shear connection to the web: The design of the shear connection for this case will not be carried out in detail here (see Chap. 2 for design of shear connections). From the *AISC Manual*, Part II, a 5/16-in pair of angles with $4^7/_8$ in A325X bolts provide 137 kips of shear resistance. This is larger than the 75 kips required for design. The final design is shown in Fig. 4.10.

11. Moment and rotation at service: The connection will not slip until the frictional capacity of the bolts (M_{slip}) is reached when the force in the plate reaches approximately 102 kips. This corresponds to a stress:

$$\sigma_{\text{slip}} = \frac{102 \text{ kips}}{9.5 \times 0.75} = 14.3 \text{ ksi}$$

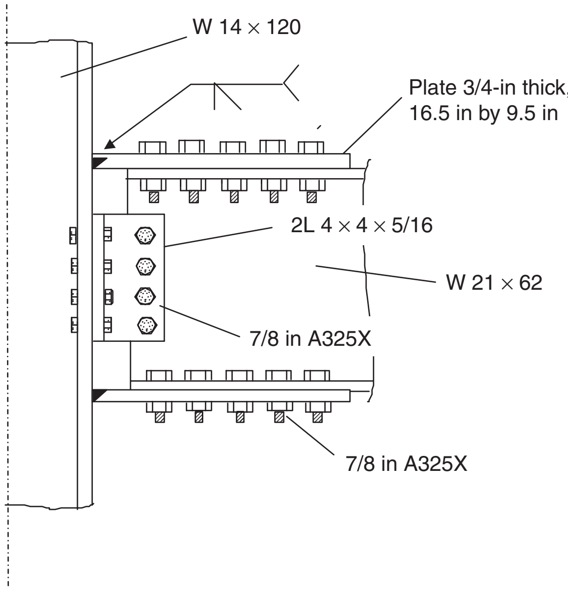

Figure 4.10 Final configuration for Example 1 connection.

For a quick check, assume a linear distribution of the force from a maximum at the column face to zero stress at the plate end. Compute the elongation of the plate:

$$\Delta_{\text{conn}} = \frac{\sigma_{\text{slip}}(L/2)}{E} = \frac{(14.3\ \text{ksi})(20\ \text{in}/2)}{29000\ \text{ksi}} = 0.0050\ \text{in}$$

Assume that the connection rotates about the center of the beam. The connection rotation is

$$\theta_{\text{conn}} = \frac{\Delta_{\text{conn}}}{(d/2)} = \frac{0.0050\ \text{in}}{(20.99\ \text{in}/2)} = 0.00047\ \text{rad}$$

The connection stiffness is

$$K_{\text{conn}} = \frac{M_{\text{slip}}}{\theta_{\text{conn}}} = \frac{(2141)(20.99)}{0.00047\ \text{rad}} = 95.6 \times 10^6\ \text{kip-in/rad}$$

The relative stiffness, assuming the beam is 24 ft. long is

$$\alpha = \frac{K_{\text{conn}} L_{\text{beam}}}{EI_{\text{beam}}} = \frac{(95.6 \times 10^6)(24 \times 12)}{(29000)(1330)} = 714\ \text{(very rigid)}$$

12. Moment and rotation at yield: For a quick check assume the plate has just begun to yield at the column face and a linear distribution of the force along

the plate. Compute the elongation of the plate and add 1/32 in for the slip of the bolts:

$$\Delta_{\text{conn}} = \frac{\sigma_{\text{plate,yield}}(L/2)}{E} + \text{slip} = \frac{(1.1 \times 50\text{ ksi}))(20\text{ in}/2)}{29000\text{ ksi}} + \frac{1}{32}$$

$$= 0.019 + 0.031 = 0.050\text{ in}$$

From this computation, elastic strains in the plate contribute approximately 40% and slip approximately 60% to the total deformation. Assume that the connection rotates about the center of the beam. The connection rotation is

$$\theta_{\text{conn}} = \frac{\Delta_{\text{conn}}}{(d/2)} = \frac{0.050\text{ in}}{(20.99\text{ in}/2)} = 0.00478\text{ rad}$$

The connection stiffness is

$$K_{\text{conn}} = \frac{M_{\text{yield,plate}}}{\theta_{\text{conn}}} = \frac{(1.1 \times 50)(9.5 \times 0.75)(20.99)}{0.00478\text{ rad}}$$

$$= 1.89 \times 10^6\text{ kip-in/rad}$$

The relative stiffness, assuming the beam is 24 ft long is

$$\alpha = \frac{K_{\text{conn}} L_{\text{beam}}}{EI_{\text{beam}}} = \frac{(1.89 \times 10^6)(24 \times 12)}{(29000)(1330)} = 14.1$$

Note that this will put this connection in the strong PR range after slip occurs. This calculation can only be considered as an estimate as from the computations above it is clear that slip is the main contributor to the deformation. The assumed slip (1/32 in) is considered a reasonable value. The moment-rotation curve for this connection indicates rigid behavior at the service level, and PR behavior between the service and ultimate load level.

13. Moment and rotation at ultimate: Assume that the plate has yielded and reached a stress halfway between the expected yield strength (55 ksi) and ultimate strength (71.5 ksi), or approximately 63.25 ksi. The corresponding moment at the column face is 9459 kip-in. At the critical location in the beam, that is, just under the first row of bolts, the moment is much higher than the beam can take (approximately 7700 kip-in based on net section and ultimate strength with no resistance factor). Thus extensive yielding and a plastic hinge in the beam would be expected before the connection reaches even its initial yield. The rotation at this level correspond to full yielding along the plate, plus 1/8 in of slip and bearing deformation top and bottom for a rotation of 9.56 mrad. The final moment-rotation curve for the connection is shown in Fig. 4.11.

Figures 4.12 and 4.13 show some typical details and variations proposed by Astaneh-Asl for this type of connections. Figure 4.12 shows a variation where the bottom flange is welded rather than bolted, while Fig. 4.13 shows a connection to the weak axis of the column.

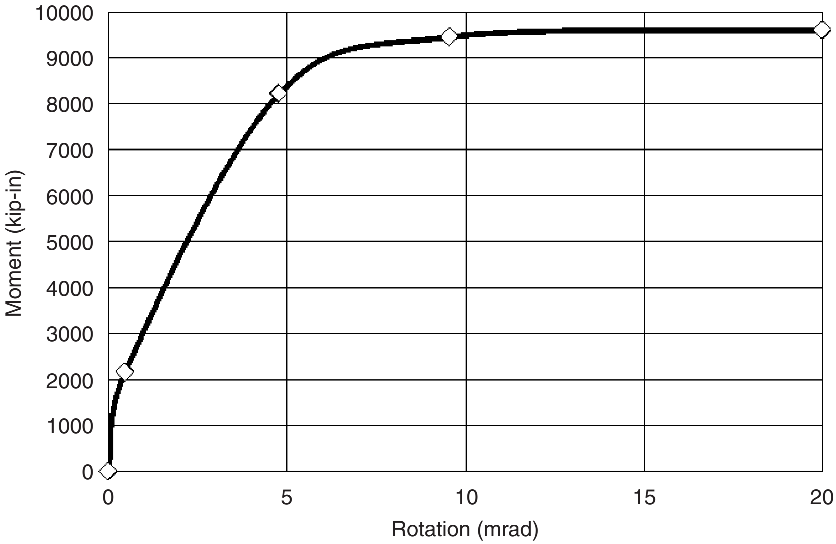

Figure 4.11 Moment-rotation curve for Example 4.1.

It is important to note that in the Example 4.1 it was assumed that the loads were well-known. In Example 4.2, it is shown that while a connection can be designed to connect similar-size members in a frame located in a high-seismic zone, the requirements can be very different.

Design Example 4.2: Design a full-strength connection between a 28-ft W 27 × 94 girder and a W 14 × 311 column. Assume the dead and live load as 0.75 kip/ft each. Both sections are A572 grade 50. Use A490X bolts. Design for seismic design category (SDC) D.

1. Determine maximum moment capacity required for the connection design:

$$M_{\text{pr}} = C_{\text{pr}} Z_x R_y F_y = \left(\frac{50 + 65}{2 \times 50}\right)(278\,\text{in}^3)\,(1.1 \times 50\,\text{ksi})$$

$$= 17{,}584 \text{ kip-in}$$

If we assume that all bending forces are transmitted through the beam flange, the force in the beam flange consistent with this moment would be:

$$F_{\text{plate}} = \frac{17584\,\text{kip-in}}{27.7\,\text{in}} = 634\,\text{kips}$$

The maximum bolt diameter will be taken as:

$$d_b \leq \frac{1}{2} b_f\left(1 - \frac{Y_t F_y}{F_u}\right) - \frac{1}{8} = \frac{1}{2}\,(9.99)\left(1 - \frac{50}{65}\right) - \frac{1}{8}$$

$$= 1.03\,\text{in (Use 1-in bolts)}$$

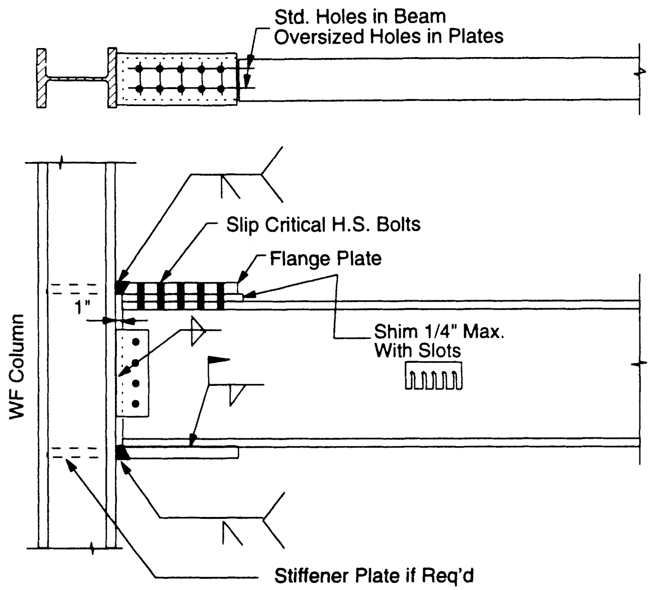

Figure 4.12 Typical CW-BB connection at the top and CW-BW connection at the bottom.

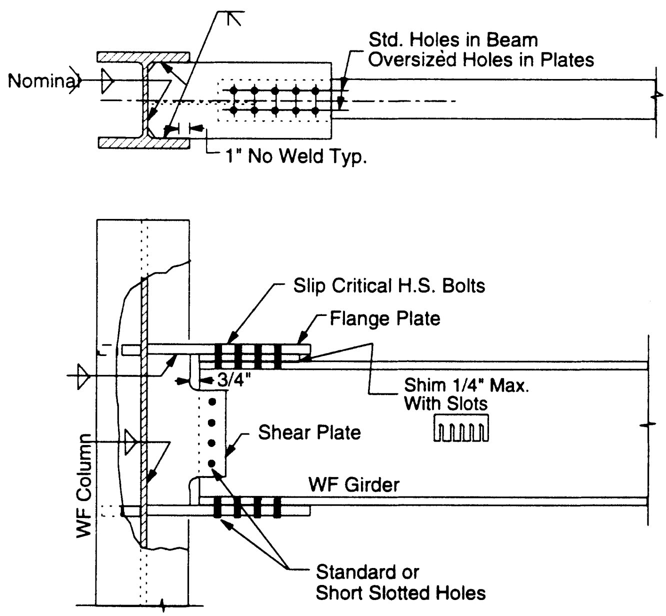

Figure 4.13 Typical CW-BB connection to weak axis of the column.

2. Check net area fracture versus gross section yielding of the girder flange by AISC (2005) Section F13. (Note: $Y_t = (F_y/F_u) = 50/65 = 0.76 < 0.8$ so $Y_t = 1.0$; for the W 27 × 94, $b_f = 9.99$ in, $t_f = 0.745$ in, and $d = 27.7$ in; holes for 1-in bolts assumed as 1.125 in in diameter):

$$F_u A_{fn} = (65)(9.99 - 2.25)(0.745) = 374.8.0 \text{ kips}$$
$$Y_t F_y A_{fg} = (1.0)(50)(9.99)(0.745) = 372.1 \text{ kips}$$

Thus the gross section governs ($F_u A_{fn} > Y_t F_y A_{fg}$) by these computations, but in reality either of them could control as the two values are very close to one another.

3. Check local buckling:
Flange:

$$\left(\frac{b}{t}\right) = 6.70 \le 0.30\sqrt{\frac{E}{F_y}} = 7.2, \text{ ok}$$

Web:

$$\left(\left(\frac{h}{t_w}\right) = 49.5 \le 3.14\sqrt{\frac{E}{F_y}} = 75.6, \text{ ok}\right.$$

4. The maximum unbraced length (L_b) for seismic design is

$L_b < 2500\, r_y F_y = 2500(1.87)/50 = 93.5 \text{ in} = 7.8 \text{ ft}$, say 7.5 ft for design

5. Estimate number of A490X 1-in bolts required for shear. Note that a 1.25 factor is used here to increase the number of bolts, as the design moment at the column face will be increased by the shear acting at the critical section:

$$n \ge \frac{1.25 M_{\text{pr}}}{\phi_v (d)(F_v A_b)} = \frac{1.25(17585)}{(1.00)(27.7)(0.79 \times 75)} = 13.3, \text{ say 14 bolts}$$

a. Determine the beam hinge location (S_h). The hinge will be located below the last row of bolts away from the column face. Assuming a bolt spacing and end distance of 3 in and a distance between the first row of holes and the column of 4 in, the hinge will be located at 25 in from the column face.

b. Compute the shear in the beam (V_h) at the location of the plastic hinges (Fig. 4.14). The actual distance between the hinges (L_v) is the total centerline distance minus the column depth minus 2 times the distance to the plastic hinge:

$$L_v = L - d_c - 2S_h = 336 - 17.1 - 2(25) = 268.9 \text{ in}$$

The shear will be computed based on assuming a $w_u = 1.2\, d + 0.5\, L = 1.2(0.75) + 0.5(0.75) = 1.275$ kip/ft. Thus:

$$V_h = \frac{2M_{\text{pr}}}{L_v} + \frac{w_u L_v}{2} = \frac{(2)(17{,}585)}{268.9} + \frac{1.275(268.9)}{2(12)} = 145.1 \text{ kips}$$

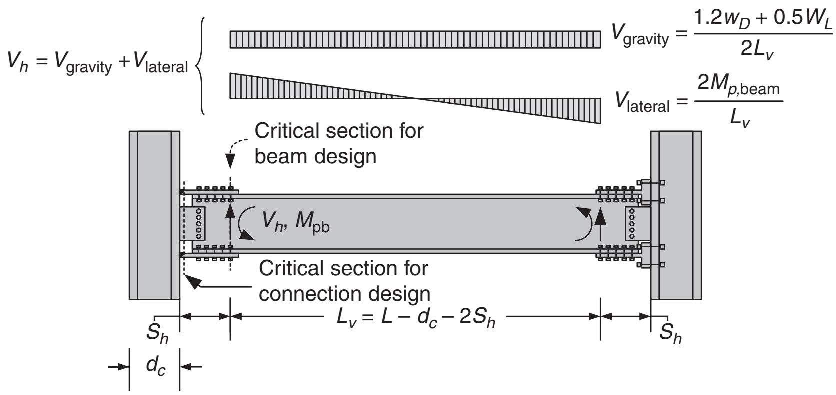

Figure 4.14 Increase in moment at connection critical section.

6. The actual moment at the face of the column (M_f) is

$$M_f = M_{pr} + V_h S_h = 17{,}585 + (145.1)(25) = 21{,}212 \text{ kip-in}$$

7. The actual force on the plate is

$$F_{plate} = \frac{21{,}212 \text{ kip-in}}{27.7 \text{ in}} = 765.0 \text{ kips}$$

8. Recheck the number of bolts

$$n \geq \frac{F_{plate}}{\phi_v(F_v A_b)} = \frac{765.8}{(1.00)(0.785 \times 75)} = 13.0, \text{ 14 bolts is ok}$$

9. Determine the size of the flange plate, assuming that the plate width will be somewhere between the widths of the beam flange (9.99 in) and the column flange (16.2 in). Assume distance between bolts is 5 in and edge distances are 4.0 in. Try a 13-in plate:

$$t_f = \frac{F_{plate}}{\phi_d F_y b} = \frac{765.8}{(1.0)(50)(13)} = 1.18 \text{ in}$$

use 1.25-in plate

10. Check net and gross areas for the plate:

$$F_{gross\ area} = R_y F_y A_{gross} = (1.1)(50)(13 \times 1.25) = 894 \text{ kips}$$
$$F_{net\ area} = F_y A_{net} = (65)(13 - 2(1.125)) \times 1.25) = 873 \text{ kips} \qquad \text{ok}$$

11. Check block shear on the plate. Assume shear failure along the bolts and tensile failure across bolt gage and $U_{bs} = 1.0$:

$$A_{gv} = 2\,(25 - 4 - 1/2)(1.25) = 51.25\ \text{in}^2$$
$$A_{gt} = (5)(1.25) = 6.25\ \text{in}^2$$
$$A_{nv} = 51.25 - 2(6.5 \times 1.125)(1.25) = 32.97\ \text{in}^2$$
$$A_{nt} = 6.25 - (1 \times 1.125)(1.25) = 4.85\ \text{in}^2$$
$$R_n = 0.6F_u A_{nv} + U_{bs}F_u A_{nt} \le 0.6F_y A_{gv} + U_{bs}F_u A_{nt}$$
$$R_n = 0.6(65)(32.75) + (1.0)(65)(4.85) \le 0.6(50)(51.25) + (1.0)(65)(4.85)$$
$$R_n = 1592\ \text{kips} \le 1853\ \text{kips}$$
$$\phi R_n = (0.75)(1592\ \text{kips}) = 1194\ \text{kips} \qquad \text{ok}$$

12. Detail the shear connection to the web: The design of the shear connection for this case will not be carried out in detail here (see Chap. 2 for design of shear connections). From the *AISC Manual*, a 5/16-in pair of angles with five 1-in A325N bolts provide 145 kips of shear resistance. This is equal to the 145 kips required for design.

13. Check connection stiffness: For a quick check assume the plate has just begun to yield at the column face and a linear distribution of the force along the plate. Compute the elongation of the plate and add 1/32 in for the slip of the bolts:

$$\Delta_{\text{conn}} = \frac{\sigma_{\text{plate,yield}}(L/2)}{E} + \text{slip} = \frac{(1.1 \times 50\ \text{ksi})(25\ \text{in}/2)}{29000\ \text{ksi}} + \frac{1}{32}$$
$$= 0.023 + 0.031 = 0.055\ \text{in}$$

From this computation, elastic strains in the plate contribute approximately 40% and slip approximately 60% to the total deformation. Assume that the connection rotates about the center of the beam. The connection rotation is

$$\theta_{\text{conn}} = \frac{\Delta_{\text{conn}}}{(d/2)} = \frac{0.055\ \text{in}}{(27.7\ \text{in}/2)} = 0.00396\ \text{rad}$$

The connection stiffness is

$$K_{\text{conn}} = \frac{M_{\text{yield, plate}}}{\theta_{\text{conn}}} = \frac{(1.1 \times 50)(13 \times 1.25)(27.7)}{0.003968\ \text{rad}}$$
$$= 6.25 \times 10^6\ \text{kip-in/rad}$$

The relative stiffness, assuming the beam is 24 ft long is

$$\alpha = \frac{K_{\text{conn}}\, L_{\text{beam}}}{EI_{\text{beam}}} = \frac{(6.25 \times 10^6)(28 \times 12)}{(29000)(3270)} = 22.1$$

Note that this will put this connection in the FR range. This calculation can only be considered as an estimate as from the computations above it is clear that slip is the main contributor to the deformation. The assumed slip (1/32 in) is a reasonable value for the first cycle. As the connection is cycled, this slip will probably increase to approximately 0.25 in and the secant stiffness will decrease correspondingly, probably putting the connection into the mid-PR range.

The rest of the checks should proceed as for Example 4.1, with additional checks for the column for (a) continuity plates, (b) doubler plates (unlikely), and (c) beam-to-column moment ratio. For the beam, additional checks for block shear and shear connection demand should be performed.

4.3.2 Column-bolted–beam-bolted connections (T-stubs)

Bolted T-stub connections were a popular connection in moment-resisting frames before field-welded connections became economical, and along with end-plate connections still represents the most efficient kind of column-bolted–beam-bolted (CB-BB) connection. The mechanistic model for this type of connection is shown in Fig. 4.5, while the possible yield and failure modes are shown in Fig. 4.8. The important conceptual difference between a CW-BB and a CB-BB is that for T stubs the springs that represent the connection to the column flange have lower strength and stiffness. This is because they represent the flexural deformations that can take place in the flanges of the tee as well as any axial deformation of the bolts to the column flange. Both of these are flexible when compared to the axial stiffness of a weld, which can be considered to be an almost rigid element. In addition for the CB-BB connections, the spring representing the bolts needs to include the prying action, which can significantly increase the force in the bolts at ultimate. Figure 4.15 shows prying action in a very flexible T stub. In this case the flexibility of the flange of the stub results in an addition prying force (Q) at the tip of the stub flange. This force increases the nominal force in the bolts above its nominal pretension value (T).

For the case of the T stub, the springs shown in Fig. 4.5 can have a wide range of strength and stiffnesses, depending primarily on the

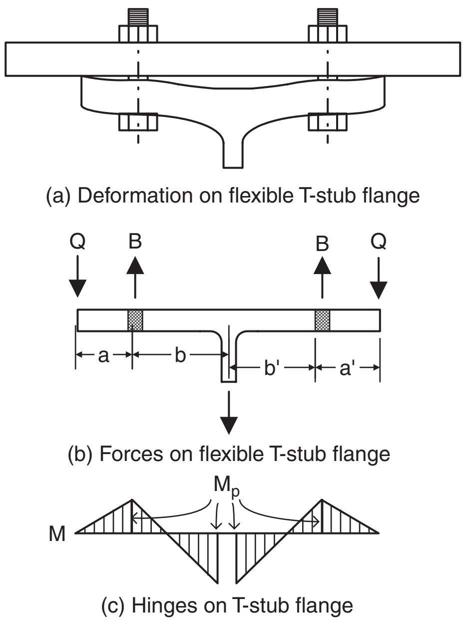

Figure 4.15 Prying action in T-stub, showing the case of a flexible flange.

thickness of the flanges and the location and size of the bolts to the column. The big advantage of this type of connection over a CW-BB one is that these springs can provide a much larger deformation capacity than a weld would. A T-stub connection can thus provide a good balance between strength, stiffness, and ductility.

The design of a T-stub connection essentially follows the same steps as for the CW-BB connections described previously (for the stem portion of the connection) with important additional design provisions for prying action, bolt tensile elongation capacity, local effects on the column flange, and bolt shear strength. The strength of the connection to the column, taking into account prying action, is limited by the following:

- *The bending strength of the flanges of the T:* This depends primarily on the thickness of the flanges and the exact location of the bolt holes.
- *The ultimate tensile strength of the stem of the T:* The net area generally governs over the gross area criteria because the width of the stem at the critical section for net area is not too different from that of the critical section for gross area.

- *The tensile strength of the bolts:* This is influenced primarily by the prying action.
- *The shear strength of the bolts:* It is difficult to fit more than 8 to 10 bolts in the stem of a conventional T (cut form a W shape) and thus large bolts may be needed.

Each of these failure modes must be checked individually and the lowest strength taken as the controlling value. Guidelines for these calculations are given in the AISC Manual (AISC, 1993), textbooks (see for example, pp. 848–856 of Salmon and Johnson, 1996), and in the standard references (Kulak et al., 1987). An excellent review of the design, including some of the numerical problems that can be encountered is given by Thorton (1985). In this chapter, Example 4.3 is based on the work of Swanson and Leon (2000, 2001), while Example 4.4 is based on unpublished work by Swanson, Rassati and Leon for AISC 358.

The effect of reversed cyclic loading on these connections is to progressively decrease the tension in the bolts to the column flange. Because of prying action, the stress range in these bolts is probably significantly larger than that calculated based on the simplified models used for design. This can result in either low-cycle fatigue failures or in fracture of the bolt due to excessive elongation.

Design Example 4.3: A rigid connection is to be designed to transfer a factored moment of 260.6 kip-ft and a factored shear of 112 kip from a W 21 × 57 beam to the flange of a W 14 × 82 column. The connection consists of T sections for moment transfer and web angles for shear transfer. All materials are A36 steel. Bolts are to be 1-in A325-N bolts. Seismic design is not required.

1. If all bending moment is carried by the tees, the force of the internal couple is

$$F \cong \frac{M_u}{d_b} = \frac{260.6 \times 12}{21.06} = 148.4 \text{ kips}$$

2. Determine the minimum number of bolts (N) required to carry the tensile force to the column flange. Ignore the prying forces for now and check later.

$$\phi R_n = \phi F_t A_b = (0.75)(90)(0.785) = 53.0 \text{ kips/bolt}$$
$$N = 148.4/53 = 2.8 \qquad \text{say 4 bolts}$$

Note that because prying forces can be large in this type of connection, it is best to have a very conservative number of bolts to the column flange. This check is used here mostly to ensure that a reasonable number of bolts are needed (that is 4 to 8 bolts rather than more, which would be hard to accommodate).

3. Determine the number of bolts (M) required to transmit the forces from the tee to the beam flanges through shear (bolts are in single shear):

$$\phi R_n = 0.75 F_v A_b = 0.75(48)(0.7856) = 28.3 \text{ kips/bolt}$$

Check bearing strength:

$$\phi R_u = \phi(2.4)dtF_u = 0.75(2.4)(1)(t)(58) \geq 104.4t$$

Shear will govern if the thickness of the plate is greater than:

$$t = 28.3/104.4 = 0.27 \text{ in}$$

This is small so bearing probably will not govern; this thickness will be checked again later.

Thus:

$$M \geq \frac{148.4 \text{ kips}}{28.3 \text{ kips}} \geq 5.2 \qquad \text{use 6 bolts}$$

4. Determine thickness (t_w) required to transmit tension on the stem of the tee (AISC 306-05):

   a. Assume plate width at critical section is approximately 9 in (total column flange width is 10.13 in). Then the required tee stem thickness for capacity controlled by gross area yielding is

$$A_g \geq \frac{T_u}{0.90F_y} \geq \frac{148.4}{0.9 \cdot 36} \geq 4.58 \text{ in}^2 \qquad \text{so } t_w = 4.58/9.00 = 0.51 \text{ in}$$

   b. Assume that the net area is given by the total width (9.00 in) minus two bolt holes ($2 \times 1.125 = 2.25$) or 6.75 in. Then the required required tee stem thickness for capacity governed by net area fracture is

$$A_n = \frac{T_u}{0.75 \times F_u} \geq \frac{148.4}{0.75 \times 58} \geq 3.41 \text{ in}^2 \qquad \text{so } t_w = 3.41/6.75 = 0.51 \text{ in}$$

   Both of them are close enough to say that t_w should be just over 0.5 in.

5. Determine the flange thickness (t_f) for the tee section. This needs to take prying action into account. A simplified mechanism for computing the additional forces due to prying action is shown in Fig. 4.16. The prying forces Q arise from the additional forces developed at the end of the T flanges as the T stub is pulled. Assuming that each side of the flange can be modeled as a two-span beam with one end fixed (at the web) and one end free to rotate (edge of T stub), the maximum forces can be calculated based on the formation of plastic hinges at both the web and the edge of the bolt. For details see Salmon and Johnson (1996, pp. 905–909).

   From this type of model, an equation for the required plate thickness can be derived. On such equation is that proposed by (Thornton, 1998):

$$t_t \geq \sqrt{\frac{4Tb'}{\phi_b wF_y(1 + \alpha\delta)}} \qquad \phi_b \geq 0.9$$

where T = force in the bolt, kips
b' = distance from the web centerline to the inside edge of the bolt, in
a' = distance from inside edge of the bolt to edge of T-stub, in

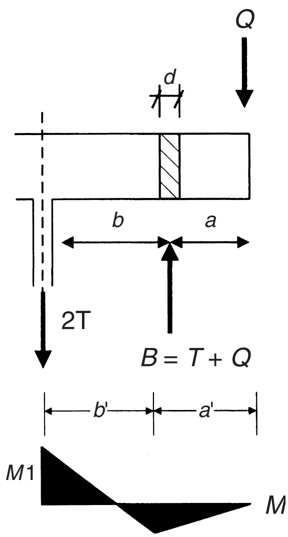

Figure 4.16 Prying action mechanisim (Salmon and Johnson, 1995).

w = tributary width of the flange, in
F_y = yield strength of the T-stub, ksi
α, δ, β = constants as defined below

Salmon and Johnson (1996), following Thorton (1985), recommend to compute β as a function of α and δ, where

$$\beta = \left(\frac{B}{T} - 1\right)\frac{a'}{b'}$$

where $\alpha = M_1/\delta\, M_2$
δ = ratio of net area at bolt line to gross area at M_1
B = maximum bolt resistance, kips

For our example, assuming a gage (g) of approximately 4 in

$$b = (g/2) - (t_w/2) = 1.75 \text{ in}$$

$$b' = b - (d/2) = 1.25 \text{ in}$$

$$\beta = \left(\frac{53.0}{37.1} - 1\right)(1.25) = 0.54$$

For purposes of this calculation the force B can be taken, as an upper bound, as the bolt capacity in tension (53.0 kips). The force T is the part of the total tension force going to each bolt (148.3/4 bolts = 37.1 kips). The value of a′ is a guess since we have not chosen a tee yet. Use α as follows:

if $\beta \geq 1$ use $\alpha = 1 \rightarrow$ large prying force

if $\beta < 1$ use α = lesser of $\frac{1}{\delta}\left(\frac{\beta}{1-\beta}\right)$ and 1.0

Estimate:

$$\delta = \frac{\text{net width at bolt line}}{\text{gross width at critical section near webface}}$$

$$\delta = \frac{4.5 - \left(1 + \frac{1}{16}\right)}{4.5} = 0.763$$

$$\frac{1}{\delta}\left(\frac{\beta}{1 - \beta}\right) = \frac{1}{0.763}\left(\frac{0.54}{1 - 0.54}\right) = 1.53 \rightarrow x = 1$$

$$t_f \geq \sqrt{\frac{4(148.4/2)1.25}{0.9 \times 9 \times 36 \times (1 + 1 \times 0.76)}} = 0.72\,\text{in}$$

Try a WT 12 × 47, $t_f = 0.875$ in, $t_w = 0.515$ in.

6. Check the prying force using the formula proposed by Salmon and Johnson (1996):

$$Q \geq T\left(\frac{\alpha\delta}{1 + \alpha\delta}\right)\left(\frac{b'}{a'}\right)$$

$$a' = a + \frac{d}{2} = \frac{b_f - g}{2} + \frac{d}{2} = \frac{9.065 - 4}{2} + 0.5 = 3.03\,\text{in}$$

$$b' = b - \frac{d}{2} = \frac{g}{2} - \frac{t_w}{2} - \frac{d}{2} = \frac{4}{2} - \frac{0.51}{2} - 0.5 = 1.25\,\text{in}$$

Taking the length of the tee section as 9 in with two holes deducted:

$$\delta \geq \frac{9 - 2\left(1 + \frac{1}{16}\right)}{9} \geq 0.76$$

$$\alpha \geq 1 \rightarrow \alpha\delta = 0.763$$

$$Q = T\left(\frac{0.763}{1 + 0.763}\right)\left(\frac{1.25}{2}\right) = 0.27\,T$$

$$T + Q = 1.27\,T$$

$$1.27 \times T_u = 1.27 \times 37.1 = 47.117 \leq \phi R_n$$

7. Recheck the thickness t_t required

$$t_f \geq \sqrt{\frac{4T \times b'}{\phi w F_{y(1-x8)}}} = 0.72\,\text{in}$$

Design is satisfactory; use WT 12 × 47 to carry tensile and compression forces.

8. Design the angles for shear transfer:

$$P_u = 112\,\text{kips}$$

The angles are in single shear so:

$$\phi R_n \geq 0.75(48)\,1\,(0.7854) \geq 28.3\,\text{kips/bolt}$$

There are six bolts so the capacity is $6 \times 28.3 = 169$ kips $> P_u$, so shear does not govern. Bearing will control in the angles and not in the column flange which is much thicker. The minimum angle thickness is therefore:

$$\phi R_n(\text{bearing}) \geq \phi(2.4F_u) \times d \times t \geq 28.3\,\text{kips}$$

$$t_{f\,\min} \geq \frac{28.3}{(0.75)(2.4)(58)(1)} \geq 0.27\text{ in}$$

$$\rightarrow \text{use } 2L \times 4 \times 4 \times 5/16 \times 1\,\text{ft-}0\text{ in}$$

Number of required bolts:

$$N = \frac{112}{28.3/\text{bolts}} = 3.95\,\text{bolts} \Rightarrow \text{use 4 bolts}$$

Check shear in net section:

$$A_{ns} = t\left[12\text{ in} - 5\left(1 + \frac{1}{16}\right)\right] = 6.6875t$$

$$\text{required } t \geq \frac{112/2}{0.75 \times 0.6 \times 58 \times 6.6575} \geq 0.32\text{ in} \Rightarrow \text{use } t = 3/8\text{ in}$$

$$\rightarrow \text{use } 2L\ 4 \times 4 \times 3/8 \times 1\,\text{ft-}0\text{ in}$$

9. Check if stiffeners in column are required: To avoid stiffeners, the column web must be checked for

a. Compression zone

(1) K 1.3—local web yielding:

$$P_{bf} = \phi(5k + t_{fb})F_{y_c} \times t_{w_c} \geq (1.0)\,(5 \times 1.625 + 0.65)(36)(0.51)$$
$$= 161.1\,\text{kips}$$

(2) K 1.4—web crippling:

$$P_{bf} = \phi 135 t_{we}^2\left[1 + 3\left(\frac{t_{fb}}{d}\right)\left(\frac{t_{we}}{t_{fe}}\right)^{1.5}\right]\sqrt{\frac{F_{ye}t_{ye}}{t_{we}}}$$

$$= 0.75 \cdot 135(0.51)^2\left[1 + 3\left(\frac{0.65}{14.31}\right)\left(\frac{0.51}{0.855^{151}}\right)\right]\sqrt{\frac{36 \cdot 0.55}{0.51}} \geq 174.39\,\text{kips}$$

(3) K 1.5—compression buckling of the web:

$$R_n = \frac{4100\, t_w^3\sqrt{F_{yc}}}{d_c} = \frac{4100(0.51)3\sqrt{36}}{11} = 267\,\text{kips}$$

Thus none of the capacities are exceeded and no stiffeners are required!

b. Tension zone:

(1) K 1.2—local flange bending:

$$P_{bf} = \phi_b 6.25 \cdot t_{fc}^2 \cdot F_{yc} = 0.9 \cdot 6.25(0.855)^2 \cdot 36 = 148.0\,\text{kips}$$

Thus P_{bf} equals the required strength and no tension stiffener are required! The final design is shown in Fig. 4.17.

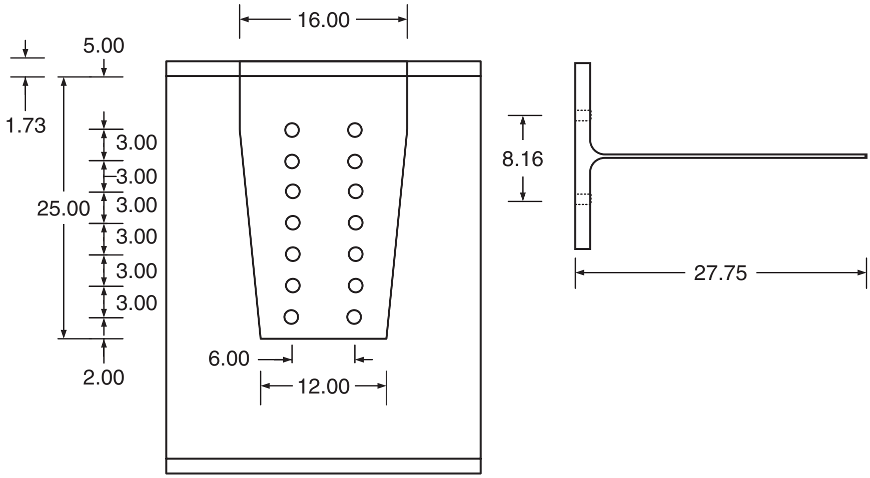

Figure 4.17 Preliminary layout for T-stub cut from W 40 × 264 beam section for Example 4.4.

Design Example 4.4: Redo Example 4.2 but for a CB-BB (T-stub) connection: Design a full-strength connection between a W 27 × 94 girder and a W 14 × 311 column. Assume the dead and live load as 0.75 kip/ft, each. Both sections are A572 grade 50. Use A490X bolts. Design for a SDC D frame.

Based on the proposed procedures for new version of AISC 358, the design of the stem will be the same as for the CW-BB connection in Example 4.2. Thus steps (1) through (8) of Example 4.2 will be valid, and this design will start with the selection of the stem thickness for the T stub, a step corresponding to step (9) in Example 4.2.

9. Determine the width of the stem (W_T), assuming that the stem width will vary between something larger than the width of the beam flange (9.99 in, say 12 in) at the beam end and somewhat less than the column flange width (16.2 in, say 16 in) at the last row of bolts (see Fig. 4.17). Assume gage between bolts is 6 in and edge distances are 2.0 in at the beam end. Assume that the distance to the first row of bolts from the column face has been increased to 5 in to eliminate any clearance problems; the length of the plate will be kept at 28 in. Note also that the Whitmore width will not govern in this case. At the critical section for net area (last set of bolts), the width of the plate is 16 in, so

$$t_f = \frac{F_{\text{plate}}}{\phi_d F_y b} = \frac{765.8}{(1.0)(50)(16)} = 0.96 \text{ in}$$

use a T-stub cut from a W 40 × 264 (t_w = 0.96 in)

10. Check net and gross areas for the plate (see Fig. 4.18 for nomenclature):

$$F_{\text{gross area}} = R_y F_y A_{\text{gross}} = (1.1)(50)(16 \times 0.96) = 845 \text{ kips}$$

$$F_{\text{net area}} = F_y A_{\text{net}} = (65)(16 - 2(1.125)) \times 0.96) = 858 \text{ kips} \qquad \text{ok}$$

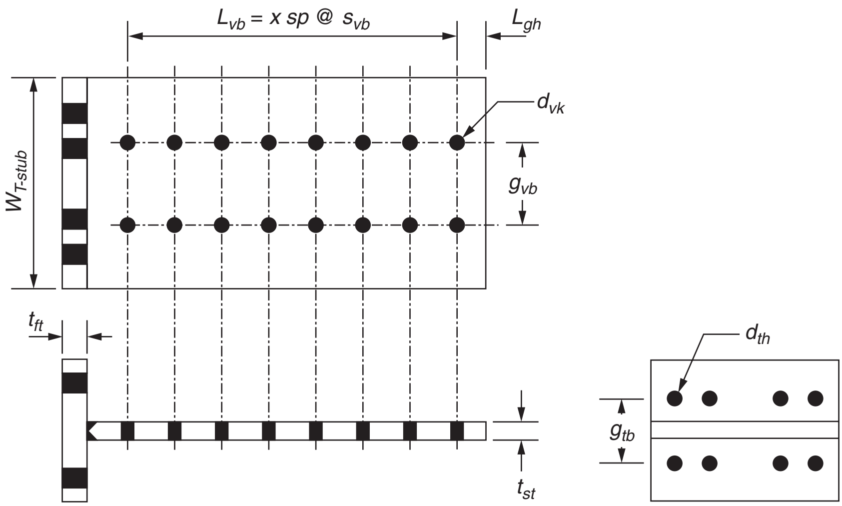

Figure 4.18 Nomenclature for T-stub design in Example 4.4

11. Check shear resistance of the bolts:

$$R_n = nA_bF_v = (14)(0.785)(75) = 824\,\text{kips}$$

Note that the shear resistance of the bolts is slightly less than the expected yielding or nominal fracture capacity of the stem [(845 kips and 858 kips, respectively from step (10)], but far larger than the actual force in the plate including the effect of shear [(765 kips from step (7), Example 4.2). In addition, this shear resistance provides more than 150% of the expected yield capacity of the beam ($R_yZ_xF_y$). Clearly extensive beam yielding and strain hardening will occur in the beam before we get even close to the critical values for the stem resistance.

12. Check block shear on the plate. As this did not govern for Example 4.2 by a large margin, and the dimension for the tensile section has increased from 5 to 6 in while the remaining dimensions stay the same, this check will be skipped for this example.

13. Detail the shear connection to the web: Same as for Example 4.2. From the *AISC Manual,* a 5/16-in pair of angles with five 1-in A325N bolts provide 145 kips of shear resistance. This is equal to the 145 kips required for design.

14. Determine the number of tension bolts required. Assuming eight bolts, the minimum bolt diameter in the absence of prying is given by

$$d_{tb} \geq \sqrt{\frac{1.25M_{pr}}{dn_{tb}\phi_n F_{nt}\left(\frac{\pi}{4}\right)}} = \sqrt{\frac{4F_f}{n_{tb}\phi_n F_{nt}\pi}} = \sqrt{\frac{4(765.8)}{(8)(0.9)(113)\pi}} = 1.00\text{ in}$$

assume $1^1/_4$-in bolt

Following the nomenclature in Fig. 4.18,

$$p = \frac{2w_T}{n_{tb}} = \frac{2(16)}{8} = 4 \text{ in/bolt}$$

$$\delta = \left(1 - \frac{d_{th}}{p}\right) = \left(1 - \frac{1.3125}{4.00}\right) = 0.672 \text{ in}$$

Assuming the gage of the tension bolts (g_{tb}) as 8 in. in order to avoid clearance problems:

$$t_{st,\text{eff}} = t_{st} + 2k_1 = 0.96 + 2(1.69) = 4.33 \text{ in}$$

$$b = \left(\frac{1}{2}\right)(g_t - t_{st,\text{eff}}) = \left(\frac{1}{2}\right)(8 - 4.33) = 1.84 \text{ in}$$

$$b' = b - \frac{1}{2}d_b = 1.84 - 0.63 = 1.21 \text{ in}$$

$$a = 1\frac{1}{2}d_b \leq 1.25b = 1.88 \text{ in} < 1.25(1.60) = 2.00, \text{ ok}$$

$$a' = a + \frac{1}{2}d_b = 1.875 + 0.625 = 2.50 \text{ in}$$

$$b_{ft} = g_{tb} + 2a = 8 + 2(1.88) = 11.75 \text{ in}$$

The flange width of the W 40 × 264 is 11.9 in. For fabricating ease, take $a = 1.88$ in and $g_t = 8.16$ in (correspondingly, $b = 1.92$ in, $b' = 1.29$ in, and a′ = 2.50 in).

15. The capacity of the tension bolts is computed as

$$\phi_t r_{nt} = A_b F_{nt} = (0.75)(1.23 \text{ in}^2)(113 \text{ ksi}) = 104.3 \text{ kip/bolt}$$

The required T-flange capacity in units of kips per tension bolts is

$$T_{\text{reqd}} = \frac{F_{pr}}{n_{tb}} = \frac{765.8 \text{ kip}}{8 \text{ bolts}} = 95.7 \text{ kip/bolt} \leq r_{nt} \qquad \text{ok}$$

16. Determine adequacy of the flange thickness. Three mechanisms can be postulated:

a. For a pure plastic mechanism in the tension flange, the required design resistance per tension bolt is

$$\phi T_1 = \frac{(1 + \delta)}{4b'}(p)(\phi_d F_y)(t_{ft}^2)$$

$$= \frac{(1 + 0.672)}{(4)(1.29)}(4.00)(1.00)(50)(1.73)^2 = 193.9 \text{ kip/bolt}$$

b. For a mixed failure mode, with a plastic mechanism followed by fracture of the bolts is

$$\phi T_2 = \frac{\phi_d r_{nt} a'}{a' + b'} + \frac{\phi_d p F_y t_{ft}^2}{4(a' + b')} \geq T_{\text{reqd}}$$

$$= \frac{(1.00)(104.3)(2.50)}{(2.50 + 1.29)} + \frac{(1.00)(4.00)(50)(1.73)^2}{(4)(2.50 + 1.29)}$$

$$= 68.8 + 39.5 = 108.3 \text{ kip/bolt}$$

c. For the limit state of bolt fracture without yielding of the tension flange, the design resistance per tension bolt is calculated as

$$\phi T_3 = \phi_d r_{nt} = (1.00)(104.3 \text{ kip/bolt}) = 104.3 \text{ kip/bolt}$$

Compute the net capacity of the tension flange is

$$\phi R_n = n_{tb}\phi T = n_{tb}\phi T_3 = (8 \text{ bolts})(104.3 \text{ kip/bolt}) = 834.4 \text{ kips} > F_{pr}$$

$$= 765.8 \text{ kips} \qquad \text{ok}$$

17. In addition to the checks above for the connection itself, the column must be checked for following limit states: (1) panel zone shear, (2) need for continuity plates, (3) local web yielding, (4) web crippling, (5) compression buckling of the web, and (6) local flange bending.

The cyclic performance of a well-designed T-stub connection is shown in Fig. 4.19. The figure shows excellent energy dissipation and stiffness to a rotation of 0.04 rad, with a decline shortly afterward due to local buckling of the beam (see Fig. 4.1).

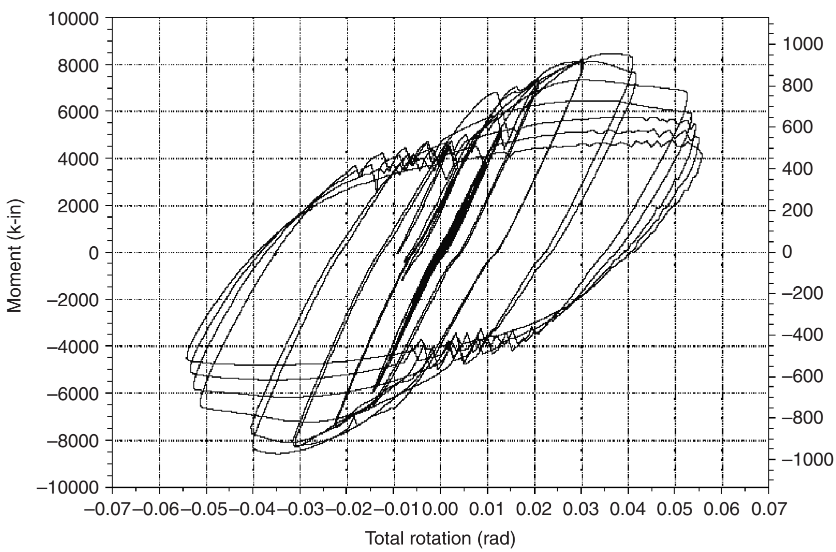

Figure 4.19 Cyclic performance of T stub.

4.3.3 End-plate connections

End-plate connections are common in some areas of the country and very popular in prefabricated metal buildings. The mechanistic behavior of an end-plate connection is very similar to that of a T stub, with the difference being that the size of the plate is longer than that of the flange of a T stub. If the plate is thin or of moderate thickness compared to the column flange, yield lines will form between the holes in the plate resulting in a plastic mechanism. Because the pattern of yield lines can be complex, the computation of the strength of the plate is not as simple as for a T stub. In the latter case, only two yield lines occur on each half of the stub, one at the bolts and one at the intersection of the flange and web (Fig. 4.17). Two typical yield-line patterns for some common end-plate configurations are shown in Fig. 4.20 (Murray and Meng, 1996, Murray and Watson, 1996). The group patterns can be very complex and not easy to determine for cases with multiple bolts in one row. Yield lines around each individual bolt, in addition to the group patterns shown in Fig. 4.16, are also possible. If the end plate is thick, the behavior will shift to that of a thick T stub. In this case the failure will be either by tension in the bolts to the column or bolt shear in the connection to the beam. In all cases, care should be exercised in not overstressing the column flanges. The strength of the column flange can be checked by a yield-line approach (Nader and Astaneh-Asl, 1992), just as for the plate itself (Fig. 4.21).

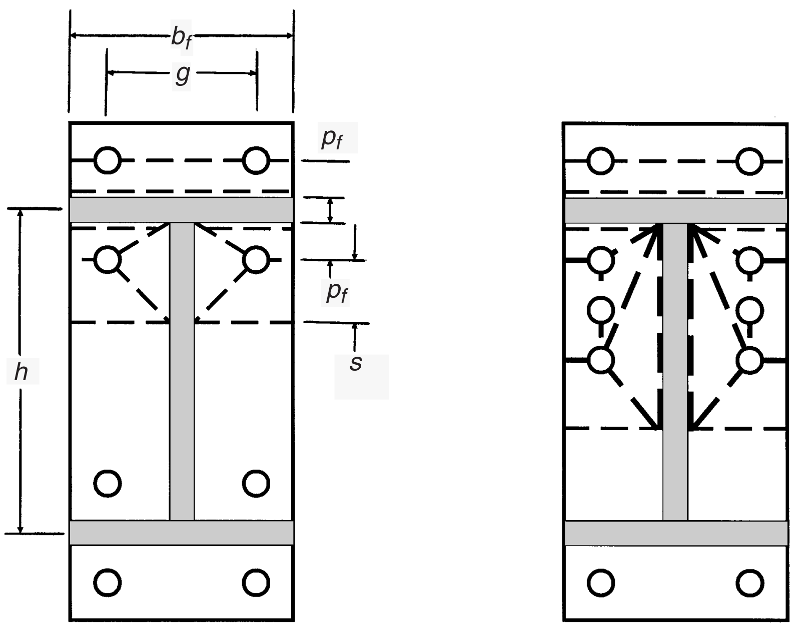

Figure 4.20 Typical yield-line patterns for end plates (Murray and Meng, 1996, Murray and Watson, 1996).

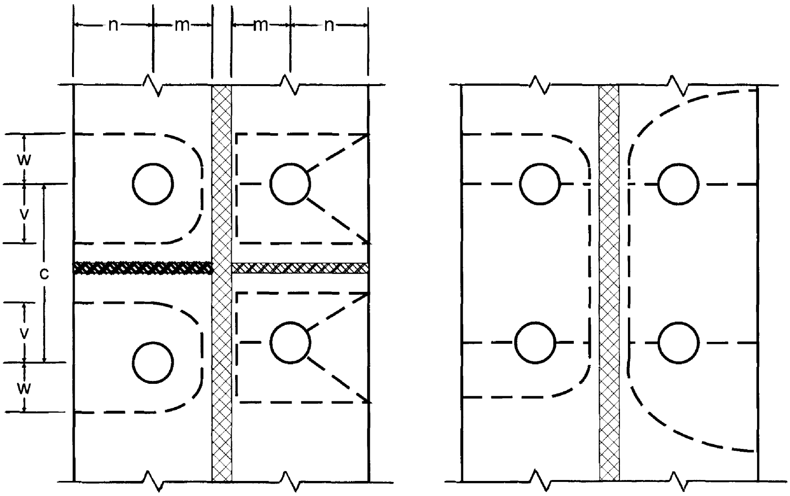

Figure 4.21 Typical yield-line patterns for column flanges with and without stiffeners (Nader and Astaneh-Asl, 1992).

An excellent review of the development of end-plate connections is given by Griffiths (1984), and detailed design guidelines and design aids for their design under monotonic loading are available (Murray, 1990). Recently Murray and Meng (1996) have suggested a direct formula for calculating the thickness, t_p, of an end plate for a four-bolt unstiffened end plate:

$$t_p = \left\{ \frac{M_u/F_{py}}{((b_f/2)[(1/p_f) + (1/s)] + (p_f + s)(2/g))(h - t_f - p_f) + (b_f/2)[(h/p_f) + (1/2)]} \right\}^{1/2}$$

$$s = \sqrt{\frac{b_f g}{2}}$$

where F_{py} is the yield stress of the end-plate material and t_p is the thickness of the plate, b is the width, p_f is the distance from the beam flange to the bolt centerline, s is the distance to the last yield line, and the subscripts b and f refer to the plate and flange, respectively.

Once the plate thickness has been selected, the actual capacity of the connection can then be calculated as

$$M_u = \left\{\left[\frac{b_f}{2}\left(\frac{1}{p_f} + \frac{1}{s}\right) + (p_f + s)\left(\frac{2}{g}\right)\right](h - t_f - p_f) + \frac{b_f}{2}\left(\frac{h}{p_f} + \frac{1}{2}\right)\right\}F_{py}\, t_p^2$$

Once this computation is made, it must be checked against the maximum capacity of the bolts. The latter is governed by prying action and can be computed based on the techniques discussed in Examples 4.3 and 4.4, or by the flowcharts from Murray shown as Figs. 4.22 through 4.24.

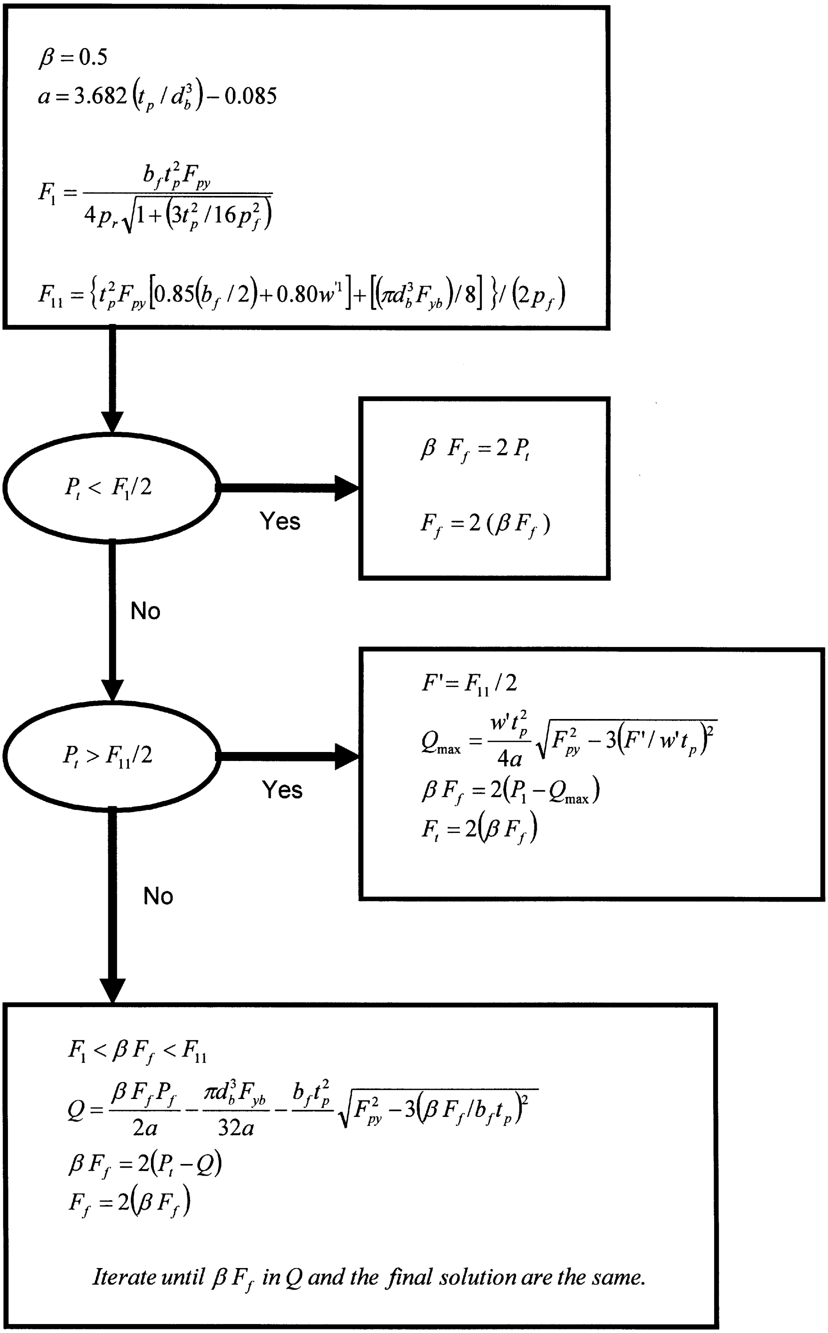

Figure 4.22 Flowchart for determining flange force for inner bolt (Murray and Meng, 1996).

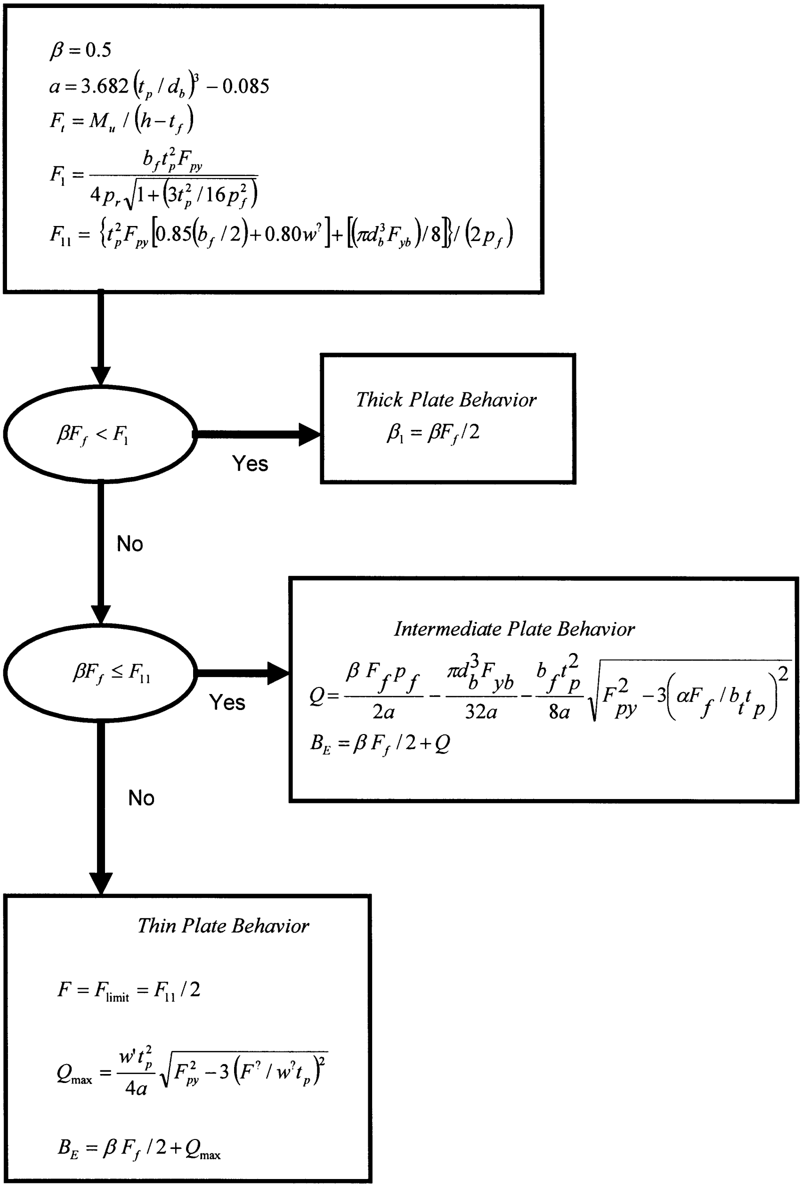

Figure 4.23 Flowchart for determining inner bolt force (Murray and Watson, 1996).

The bolt forces must be checked separately for the interior and exterior row of bolts and the prying action added to the most critical case. Note also that the equations are based on an assumed yield-line pattern. Different yield lines have been treated by other authors. The reader is

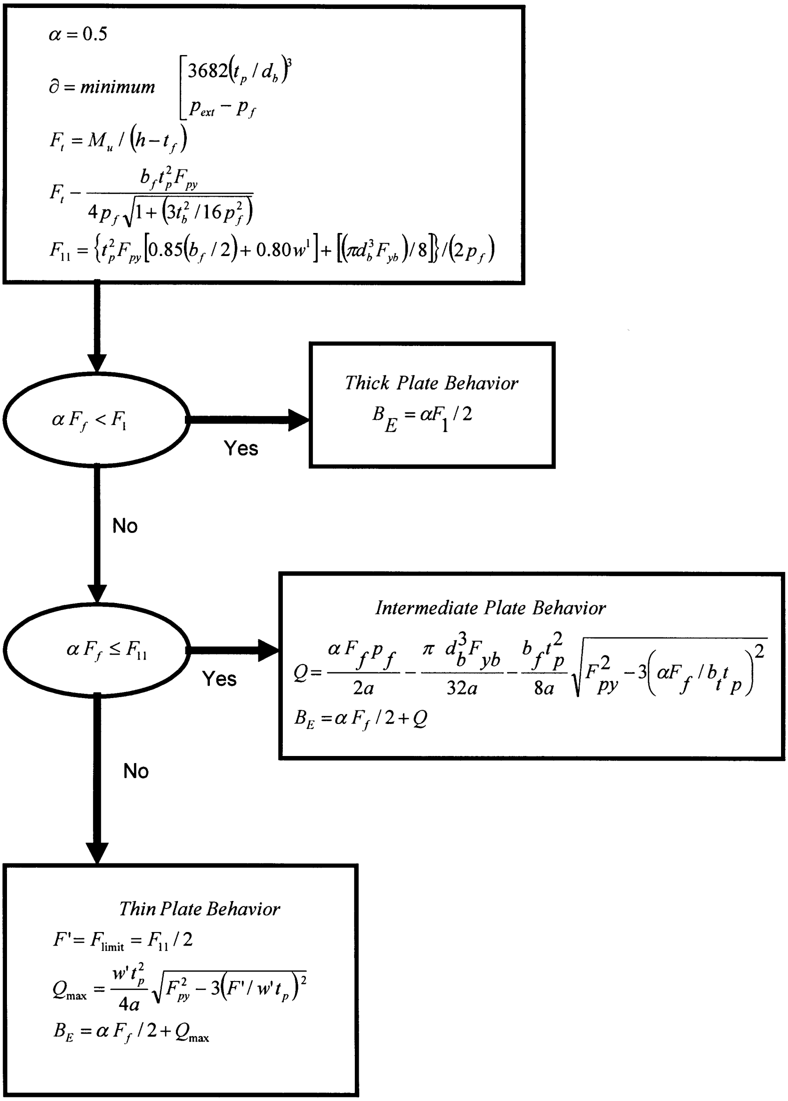

Figure 4.24 Flowchart for determining outer bolt force including prying action (Murray and Watson, 1996).

referred to Murray and Kukreti (1988), Murray (1990), Murray and Abel (1992), and Astaneh-Asl et al. (1991) for additional details.

Design Example 4.5. Determine the required end-plate thickness and bolt size for a four-bolt extended unstiffened moment end-plate connection. Use A572 grade 50 for both the beam and end plate, and A325 for the

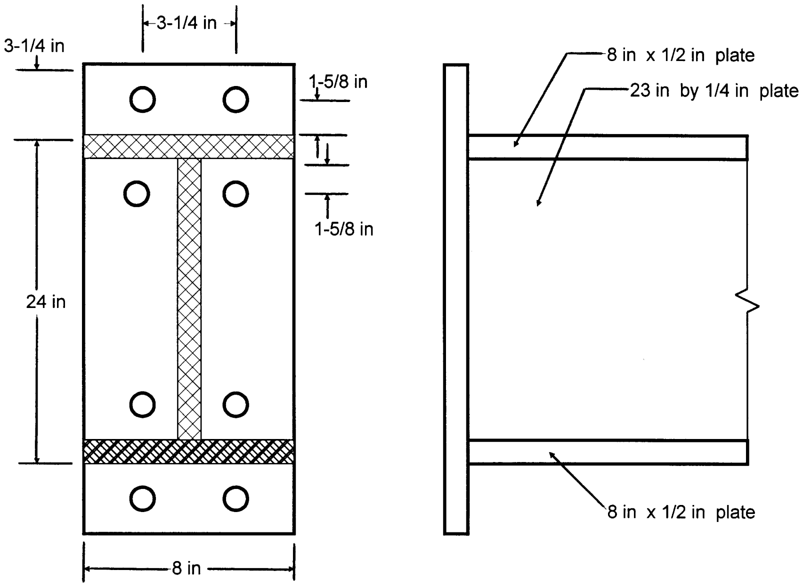

Figure 4.25 End-plate example.

bolts. The factored design moment is 225 kip-ft. See Fig. 4.25 for details of the beam and end-plate sizes.*

1. Calculate s and the required end-plate thickness t_p. Using the equations above and the dimensions in Fig. 4.24:

$$s = \frac{1}{2}\sqrt{b_t g} = \frac{1}{2}\sqrt{8(3.25)} = 2.55 \text{ in}$$

$$t_p = \sqrt{\frac{225\,(12)/50}{(8/2)[(1/1.625) + (1/2.55)] + [1.625 + 2.55)(2/3.25)](24 - 2.125) + (8/2)[(1/2) + (24/1.625)]}}$$

$$t_p = 0.51 \text{ in} \qquad \text{use } t_p = \frac{5}{8} \text{ in}$$

2. Determine the critical moment, M_{crit}, as the smallest of the moment capacities of the end plate, M_{plate}, due to the formation of yield lines and failure of the bolts, M_{bolt}, due to prying action. The moment capacity governed by the end plate, M_{plate}, is calculated as follows:

*This example is from Murray and Abel (1992), with corrections in 1994 and 1995. It was kindly provided by Dr. T. M. Murray of the Virginia Polytechnic Institute.

$$M_{\text{plate}} = \frac{F_{py}t_p^2}{12}\left\{\left[\frac{b_f}{2}\left(\frac{1}{p_f}+\frac{1}{s}\right)+(p_f+s)\left(\frac{2}{g}\right)\right](h-p_t)+\frac{b_f}{2}\left(\frac{1}{2}+\frac{h}{p_f}\right)\right\}$$

$$= \frac{50(0.625)^2}{12}\left\{\left[\frac{8}{2}\left(\frac{1}{1.625}+\frac{1}{2.55}\right)+(1.625+2.55)\left(\frac{2}{3.25}\right)\right](24-2.125)\right.$$

$$\left.+\frac{8}{2}\left(\frac{1}{2}+\frac{24}{1.625}\right)\right\}$$

$$= 334.4 \text{ kips-ft}$$

To compute the capacity of the connection based on the bolts, a bolt trial size must be chosen. Assume ¾-in A325 bolts. The force yield capacity, P_t, of each pair of bolts, based on $F_y = 44$ ksi for the bolt material, is

$$P_t = 2 \times 44 \text{ ksi} \times 0.4418 \text{ in}^2 = 38.9 \text{ kips}$$

From the flowchart given in Fig. 4.22, determine the force in the inner bolts:

$$F_1 = \frac{b_f t_p^2 F_{py}}{4\,p_f\sqrt{1+(3t_p^2/16\,p_f^2)}}$$

$$= \frac{8\,(0.625^2)50}{4(1.625)\sqrt{1+[3(0.625^2)/(16\times 1.625^2)]}} = 23.7 \text{ kips}$$

$$w^1 = \frac{b_f}{2} - \left(d_b + \frac{1}{16}\right) = \frac{8}{2} - \left(0.75 + \frac{1}{16}\right) + 3.1875 \text{ in}$$

$$F_{11} = \frac{t_p^2\,F_{py}\,[0.85(b_f/2)+0.80w] + [(\pi d_b^3 F_{yb})/8]}{2p_f}$$

$$= \frac{0.625^2\,(50)[0.85(8/2)+(0.80)3.1875] + \{[\pi 0.75^3(81)/8]\}}{2(1.625)} = 39.9 \text{ kips}$$

3. Determine the force in the flange, F_f. Since $P_t > F_{11}/2$, we have thin-plate behavior. Therefore

$$a = 3.682\left(\frac{t_p}{d_b}\right)^3 - 0.085 = 3.682\left(\frac{0.625}{0.75}\right)^3 - 0.085 = 2.05 \text{ in}$$

$$F' = \frac{F_{11}}{2} = 20.0 \text{ kips}$$

$$Q_{\max} = \frac{w' t_p^2}{4a}\sqrt{F_{py}^{\,2} - 3\left(\frac{F'}{w' t_p}\right)^2} = \frac{3.1875\,(0.625)^2}{4\,(2.05)}$$

$$\times\, 50^2 - \frac{320.0}{3.1875 \times (0.625)^2}$$

$$= 7.12 \text{ kips}$$

$$F_t = \frac{M_u}{h - t_f} = \frac{225(12)}{24} - 0.5 = 114.9 \text{ kips}$$

$$ßF_f = 2\,(P_t - Q_{\text{max}}) = 2(38.9 - 7.12) = 63.6 \text{ kips}$$

$$F_f = 2ßF_f = 2(63.6) = 127.2 \text{ kips}$$

$$M_{\text{bolt}} = F_f(h - t_f) = \frac{127.2}{12}(24 - 0.5) = 249.1 \text{ kip-ft}$$

$$M_{\text{crit}} = M_{\text{bolt}} = 249.1 \text{ kip-ft}$$

since

$$M_{\text{plate}} = 334.4 \text{ kip-ft} > M_{\text{bolt}}$$

4. Determine the inner end-plate behavior. From Fig. 4.23 and the values given previously for a, F_f, F_1, and F_{11}, $ßF_t = 0.5\,(114.9) = 57.5$ kips $> F_{11} = 39.9$ kips, inner end-plate behavior is thin-plate behavior. From the flowchart in Fig. 4.23, the inner bolt force is

$$B_f = \frac{ßF_t}{2} + Q_{\text{max}} = \frac{0.5\,(114.9)}{2} + 7.12 = 35.8 \text{ kips}$$

5. Check the outer bolt force. From Fig. 4.24, the outer bolt force is equal to $\alpha F_f/2 = 28.7$ kips, so the inner bolts control at $B_f = 35.8$ kips.

6. Checking the bolt diameter:

$$d_b = \sqrt{\frac{2B_{\text{max}}}{\pi F_1}}$$

$$= \sqrt{\frac{2(35.8)}{\pi(44)}} = 0.72 \text{ in} \rightarrow \text{use } \tfrac{3}{4}\text{-in-diameter bolt as assumed}$$

While many models of end-plate behavior exist, there is relatively little work on the design of end plates for cyclic loads (Ghobarah et al., 1992; Nader and Astaneh-Asl, 1992). Nader and Astaneh-Asl (1992) reviewed the available data and proposed design provisions. They listed plastic yield-line formation in the end-plate and column flange bending as the most desirable failure modes.

For developing design provisions the end plate can be separated into two T stubs (Packer and Morris, 1977), which results in a very similar approach to design to that developed in the previous section. The design forces can be calculated from free-body diagrams such as those shown in Fig. 4.1*a* and 4.16. Replacing a with n and b with v in Fig. 4.16 to follow the nomenclature in Astaneh-Asl (1995) and using appropriate resistance ϕ_b and material overstrength factors α to satisfy capacity design criteria, equilibrium of forces between the force in the plate F_{ep}, and the force in the beam flange, F_{fb}, gives

$$\phi_b F_{ep} = \mu\phi_b F_{fb}$$

$$\phi_b \frac{(2)(M_v + M_{v''})}{v} = \mu\, \phi_b \frac{M_{pb}}{(d_b - t_{fb})}$$

$$\phi_b \frac{(2)(M_v + M_{v'})}{v} = \frac{M_{yc}}{(d_b - t_{fb})}$$

$$M_v = \frac{b_p t_p^2}{4} f_y \qquad \text{and} \qquad M_{v'} = \frac{[(b_p - (N/2)D']\, t_p^2}{4} f_y$$

where α is the ratio of the connection yield moment to the plastic moment capacity of the beam, ϕ_b is the resistance factor for the bolts, F_{ep} is the force in the flange corresponding to yielding of the end plate, F_{fb} is the axial force in the beam flange corresponding to the plastic capacity of the beam, b_p is the width of the end plate, t_p is the thickness of the end plate, N is the number of bolts, and D' is the diameter of the bolt holes.

For end plates, it is recommended that two yield-line patterns be checked on the column flange (Fig. 4.21). One consists mostly of straight lines and the other incorporates curved ones (Packer and Morris, 1977; Astaneh-Asl and Nader, 1992).

$$F_{f1} = t_{fc}^2 f_y \left\{ \frac{n}{v} + \frac{n}{m} + (n - 0.5D')\left(\frac{1}{v} + \frac{1}{m}\right) + \pi + \pi \sec^2\left[\tan^{-1}\left(\frac{2}{\pi} \ln \frac{v}{m}\right)\right]\right\}$$

$$F_{f2} = t_{fc}^2 f_y \left[\left(\frac{1}{v} + \frac{1}{w}\right)(2m + 2n - D') + \frac{(2v + 2w + -D')}{m}\right]$$

$$w = \sqrt{[m(m + n - 0.5D')]}$$

To ensure that no out-of-plane bending occurs (Astaneh-Asl and Nader, 1992):

$$\phi_b F_{fb} = \phi_b \frac{M_{pb}}{(d - t_{fb})} \leq \begin{cases} \phi_b\, F_{f1} \\ \phi_b\, F_{f2} \end{cases}$$

$$\begin{cases} \text{if } \dfrac{b_p}{b_{fc}} = 1.0 & \text{then } t_{fc} \geq t_p \\ \text{if } \dfrac{b_p}{b_{fc}} = 0.5 & \text{then } t_{fc} \geq 0.65\, t_p \end{cases}$$

where M_{pb} is the plastic capacity of the beam, t_{fb} is the thickness of the beam flange, d is the distance between flange centerlines, and b_{fc} is the width of the column flange. Interpolation is permitted between b_p/b_{fc} values of 1.0 and 0.5.

As noted earlier, the publication of the AISC 358 document (AISC 358) has introduced very clear guidelines for the seismic design of steel connections. The reader is referred to that document for extensive design procedures and examples for end plate connections that reflect the most recent research results in that area (Sumner and Murray, 2002).

4.3.4 Flexible PR connections

The connections described in the previous sections all fall in the category of full-strength, partially restrained connections. With respect to stiffness, these connections have high initial stiffness and can probably be analyzed as rigid connections for service loads. There are a number of other common steel connections, primarily the top-and-seat angle, with and without stiffeners, that offer partial-strength, partial-restraint behavior. Design examples for this type of connection are available in the literature [see pp. 9-253–9-261 of the *Manual of Steel Construction,* LRFD (AISC, 1993), for example] and will not be covered here. In most cases these connections cannot provide sufficient lateral stiffness to resist large wind or earthquake loads unless all the connections in the structure are of this type and the effect of the slab is taken into account (Fig. 4.26) or the angles are stiffened (Fig. 4.27). For the design of this type of PR composite connections, shown in Fig. 4.26, the reader is referred to Chapter 23 of Chen (1995).

4.4 Considerations for Analysis of PR Frames*

The common practice for analysis of multistory frames assumes that joints are rigid and beam and columns intersect at their centerline. Using this method, there is no allowance for connection and panel zone flexibility, the spans of the beam and columns are overestimated, and the joints have no physical size and are reduced to a point. Since the PR behavior of most connections was recognized early, several modifications have been proposed to classical linear analysis techniques to account for connection flexibility. The first attempts involved modifying the slope-deflection method by adding the effect of linear rotational springs at beam ends (Batho and Rowan, 1934; Rathbun, 1936). Johnston and Mount (1941) gave a complete listing of coefficients to be used in the slope-deflection method including

*This section is reproduced from Chap. 5 of Background Reports: "Metallurgy, Fracture Mechanics, Welding, Moment Connections and Frame System Behavior," FEMA-288, FEMA, Washington, DC, March 1997.

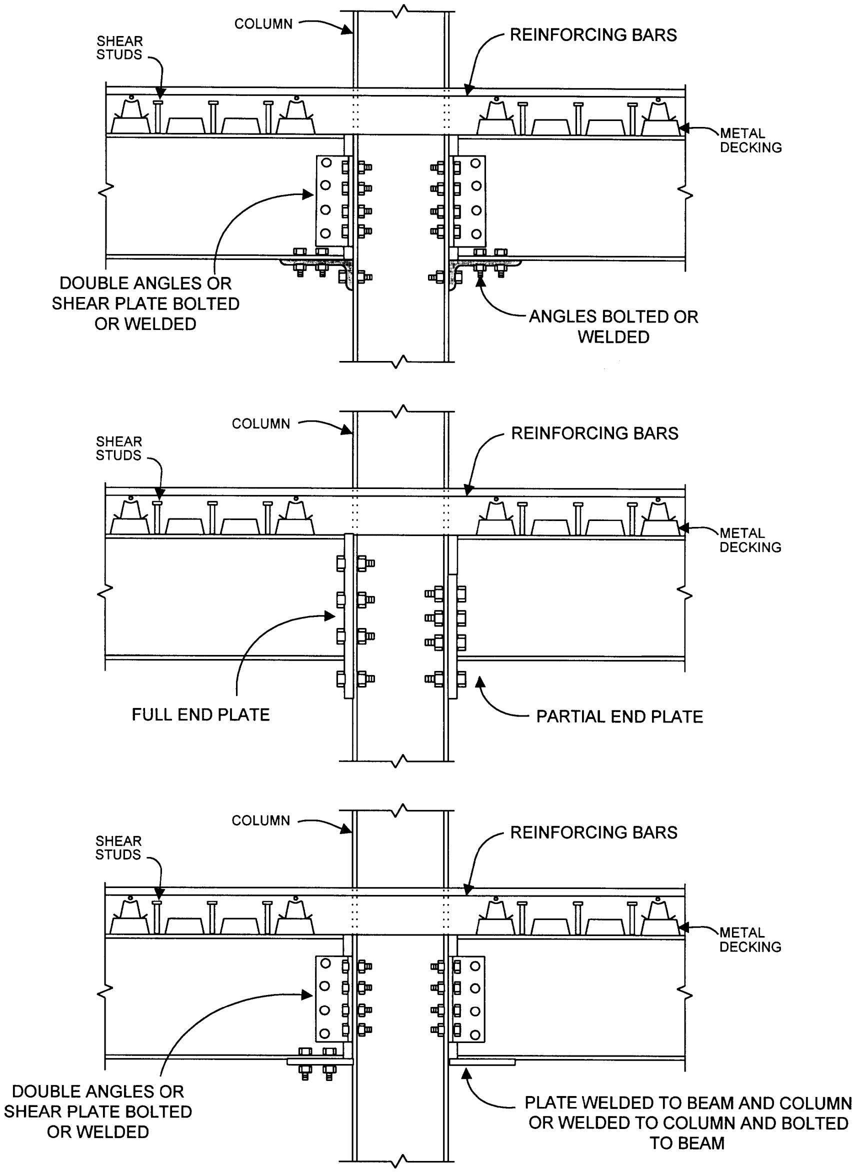

Figure 4.26 Flexible composite PR connections.

both the flexibility of the connections and the finite widths of the members. These methods were for hand calculations, and thus were limited to the analysis of relatively small structures. In an exception to this, Sourochnikoff (1950) used the beam-line method along with experimental results obtained by Rathbun (1936) to compute the

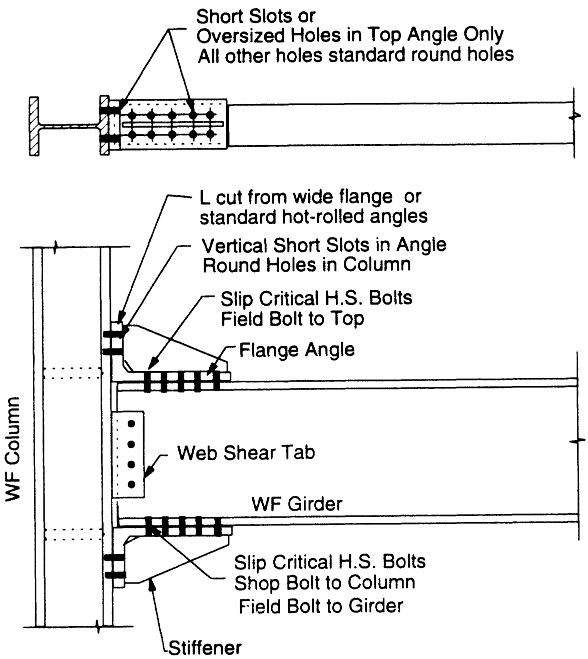

Figure 4.27 Stiffened seat connection (Astaneh-Asl, 1995).

nonlinear cyclic response of a one-story one-bay partially restrained frame. Monforton and Wu (1963) incorporated linear connection flexibility into the computerized direct stiffness method. This development permitted the analysis of large structures, but was still limited to linear analysis where the connections have constant stiffness. Lionberger and Weaver (1969) published the results from a program that performed fully dynamic lateral load analysis on plane frames. The connections in their program were modeled by a nondegrading bilinear model, which included the sizes of the rigid panel zones. Moncarz and Gerstle (1981) used a nondegrading trilinear model to analyze steel partially restrained frames subjected to lateral load reversals. When the first databases for connections were developed (Frye and Morris, 1975; Ang and Morris, 1984; Nethercot, 1985; Kishi and Chen, 1986), nonlinear expressions for moment-rotation curves became widely available. This led to the development of numerous computer programs that modeled the nonlinear behavior of PR connections. Shin (1991) devised hysteresis rules for nonsymmetrical composite connections with degradation of the unloading stiffness based on the maximum attained rotation, and implemented them in a dynamic nonlinear plane-frame analysis program.

The dynamic performance of frames incorporating partially restrained composite connections has been studied by Astaneh-Asl, Nader, and Harriot (1991), Nader and Astaneh-Asl (1992), and Leon and Shin (1995). The numerical results obtained by Leon and Shin (1995) using a modified trilinear degrading model showed excellent agreement with test results from a two-story, two-bay, half-scale frame. The analytical studies for PR frames showed good seismic performance for ground motions expected in zones of low to moderate seismicity. In particular, they showed less problems with local buckling of members and equal or better energy dissipation capacity than rigid frames. In addition, these studies showed that the lateral drifts of PR frames were within ±20% of those of companion rigid frames when four-, six-, and eight-story frames were subjected to the El Centro, Parkfield, and Pacoima ground motions. The results of these studies confirmed those of Nader and Astaneh-Asl (1992), and verified their shake table results. Further verification of the good performance can be found in the work of Osman et al. (1993) who presented the analysis results for eight-story frames with end-plate connections and flexible panel zones of various thickness.

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Chapter

5

Seismic Design of Connections

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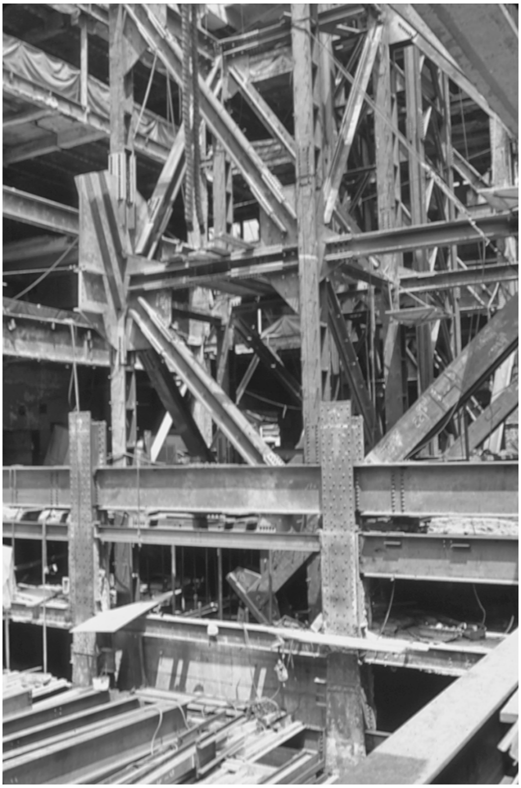
5.1 Special Design Issues for Seismic Design

The structural design philosophy for most loading conditions, such as gravity loads due to everyday dead and live loads or expected wind



(*Courtesy of The Steel Institute of New York.*)

loadings, is that the structural system, including the connections, resist the loads essentially elastically, with a safety factor to account for unexpected overloading within a certain range. The parallel philosophy for resisting earthquake-induced ground motions is in striking contrast to that for gravity or wind loading. This philosophy has evolved over the years since the inception of earthquake-resistant structural design early in the twentieth century, and continues to develop as engineers learn more about the performance of structures subjected to strong earthquakes. The present general philosophy for seismic design has been most succinctly stated in the *Bluebook* of the Structural Engineers of California (SEAOC, 1999) for a number of years. The document states this approach as the following:

> Structures designed in conformance with these Requirements should, in general, be able to:
>
> Resist a minor level of earthquake ground motion without damage.
>
> Resist a moderate level of earthquake ground motion without structural damage, but possibly experience some nonstructural damage.

> Resist a major level of earthquake ground motion—of an intensity equal to the strongest earthquake, either experienced or forecast, for the building site—without collapse, but possibly with some structural as well as nonstructural damage.
>
> It is expected that structural damage, even in a major design level earthquake, will be limited to a repairable level for most structures that meet these Requirements. In some instances, damage may not be economically repairable. The level of damage depends upon a number of factors, including the intensity and duration of ground shaking, structure configuration type of lateral force resisting system, materials used in the construction and construction workmanship.

It is clear, then, that when subjected to a major earthquake, buildings designed to meet the design requirements of typical building codes, such as the International Building Code (IBC, 2006), are expected to damage both structural and nonstructural elements. The intent of the building code under this scenario is to avoid collapse and loss of life. Because of the economic impact, structural design to resist major earthquake ground motions with little or no damage has been limited to special buildings, such as postdisaster critical structures (for example, hospitals, police, and fire stations) or structures that house potentially hazardous materials (for example, nuclear power plants).

Structural design for large seismic events must therefore explicitly consider the effects of response beyond the elastic range. A mechanism must be supplied within some elements of the structural system to accommodate the large displacement demand imposed by the earthquake ground motions. In typical applications, structural elements, such as walls, beams, braces, and to a lesser extent columns and connections, are designed to undergo local deformations well beyond the elastic limit of the material without significant loss of capacity. Provision of such large deformation capacity, known as *ductility,* is a fundamental tenet of seismic design. Note that new technologies (for example, base isolation and passive energy dissipation) have been developed to absorb the majority of the deformations and, therefore, protect the "main" structural elements from damage in a major earthquake. Such applications are gaining increasing application in areas of high seismicity. Addressing such systems is beyond the scope of this text, which will focus on the seismic design of steel connections in typical applications.

In most cases, good seismic design practice has incorporated an approach that would provide for the ductility to occur in the members rather than the connections. This is especially the case for steel frame structures, where the basic material has long been considered the most ductile of all materials used for building construction. The reasons for this approach include the following:

- The failure of a connection between two members could lead to separation of the two elements and precipitate a local collapse.
- The inelastic response of members is more easily defined and more reliably predicted.
- The inelastic action of steel members generally occurs at locations where the distribution of strain and stress does not induce constraint that could lead to a state of triaxial tension. Under certain circumstances, these connections induce significant constraint that inhibits material yielding.
- Local distributions of strain and stress in connections can become quite complicated, and be very different from simple models typically used in design.
- Connection failures in frame structures could jeopardize the stability of the system by reducing the buckling restraint provided to the building columns.
- The repair of connection damage may be more difficult and costly than replacing a yielded or buckled member.

Building codes have incorporated this philosophy into their seismic design requirements for a number of years. The most common method employed to incorporate this approach has been to require that the connections be designed to resist the expected member strength of the connecting elements, or the maximum load that can be delivered to the connection by the system. This implies that a conscious effort has been made by the designer to preclude the connections from undergoing severe inelastic demands. As such, a strength-based design approach as employed by the latest codes [for example, 2006 IBC and 2003 National Earthquake Hazards Reduction Program (NEHRP) provisions] is a much more direct and fundamental procedure than allowable stress methods that were previously followed. Seismic design of steel structures using LRFD is clearly a more rational, consistent, and transparent approach. As such, the 2005 AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2005) is based primarily on an LRFD approach. Connection design procedures in this document are based on Chapter J, "AISC Specification for Structural Steel Buildings" (AISC, 2005).

The AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2005) include a number of requirements that are intended to ensure that this philosophy can be realized in the actual seismic performance. For example, the provisions require that the expected (rather than the nominal) yield strength of the materials be considered in comparisons of relative strengths between various members and/or connections. This term, R_y, ranges from 1.1 to 1.6 depending on the material specification chosen.

Other design approaches intend for the connections themselves to absorb substantial energy and provide major contributions to the displacement ductility demand. Examples of such a system would include both fully restrained (FR) and partially restrained (PR) connections in moment-resisting frames. To properly incorporate these elements in seismic design requires a much greater level of attention than for standard connection design or for moment connections subjected only to typical static loads. In addition to typical strength design requirements, such connections should take factors such as the following into account:

- Toughness of joining elements in the connections, including any weldments.
- High level of understanding of the distribution of stress and strain throughout the connection.
- Elimination (or at least control) of stress concentrations.
- Detailed consideration of the flow of forces and the expected path of yielding in the connection.
- Good understanding of the properties of the materials being joined at the connection (for example, through-thickness, yield-to-tensile ratio).
- The nature of the connection demands being high-strain, low-cycle fatigue versus low-strain high-cycle fatigue typical of other structural applications such as bridges.
- The dynamic nature of the response which induces strain rates well below impact levels.
- The need for heightened quality control in the fabrication, erection, and inspection of the connection.

While these types of considerations are particularly critical for connections where inelastic response is anticipated, it also behooves the designer to take factors such as these into account for all connections of the seismic force resisting system.

In the AISC *Seismic Provisions*, all connections in the lateral force-resisting system are required to meet a number of basic design requirements, which go beyond those required of joints in typical steel connections. For bolted connections, the design of bolted joints require the following:

- All joints must use fully tensioned, high-strength bolts.
- Bearing design values are allowed, within the limits of the lower nominal bearing strength, $2.4dtF_u$.

- Bolted joints are not to be used in combination with welds in the same force component of the connection.
- Design of bolted joints should be such that nonductile modes do not control the inelastic performance.

For welded joints, the requirements include:

- Provision of approved welding procedure specifications that meet American Welding Society (AWS) D1.1 and are within the parameters established by the filler-metal manufacturer.
- All welds must have a Charpy V-notch (CVN) toughness AWS classification or manufacturers certification of 20 ft-lb at −20°F. For welds noted to be demand critical, an additional toughness of 40 ft-lb at 70°F must be demonstrated.
- In areas of large expected strain referred to as *protected zones*, discontinuities created by fabrication or other erecting operations are not permitted, in an effort to avoid premature fracture.

5.2 Connection Design Requirements for Various Structural Systems

Proper system selection is a critical element in successful seismic design. Various systems, such as fully and partially restrained moment-resisting frames, concentrically braced frames, and eccentrically braced frames, are addressed in the AISC *Seismic Provisions*. These provisions have specific requirements for the different structural systems that address connection design.

For moment-frame systems, special moment frame (SMF) and intermediate moment frame (IMF) connections have specified values for both inelastic deformation and strength capacities, since it is expected that these connections will absorb substantial energy during the design earthquake. Deformation capacities are to be demonstrated by qualified cyclic testing of the selected connection type. At the minimum acceptable drift deformation angle (0.04 rad for SMF, 0.02 rad for IMF), the provisions require that the nominal beam plastic moment, M_p, be reached unless local buckling or a reduced beam approach is followed, in which case the value is reduced to 0.8 M_p. The minimum beam shear connection capacity is defined as resisting a combination of full-factored dead load, a portion of the live and snow load (if any), and the shear that would be generated by the expected moment capacity (including R_y) of the beam due to seismic actions. Finally, for SMF, the joint panel zone shear is required to have a capacity able to resist the actions generated by the hinging of the beams framing into the connection. For ordinary moment frames

(OMF), the strength requirement is similar, but there is no deformation limit to 0.01 rad. No specific joint panel zone requirements are defined for OMF systems.

The design requirements for PR connections in SMF and IMF are similar to those required for FR connections as described previously. For OMF structures, a set of requirements are provided to ensure a minimum capacity level of 50% of that of the weaker connected member, and that connection flexibility is considered in the determination of the overall frame lateral drifts.

The newest moment frame system to be added to the AISC *Seismic Provisions* is the special truss moment frame (STMF). The system was developed by Professor Subhash Goel and his students at the University of Michigan (Itani and Goel, 1991; Goel and Itani, 1994; Basha and Goel, 1994). As with other steel systems, the concept of the STMF is to focus the inelastic behavior in specific elements of the truss, known as the *special segment*. The connections between the various elements of the truss and between the truss and the frame columns are designed to have a strength sufficient to develop the expected yield force and required deformation level of the special segments. The connection design requirements of AISC *Seismic Provisions* are similar for both special concentrically braced frames (SCBF) and ordinary concentrically braced frames (OCBF). For OCBF, the connections that are part of the bracing system must meet the lesser of the following:

- The nominal axial tensile strength of the bracing member, including R_y.
- The maximum force that can be transferred to the brace by the remainder of the structural system. An example of how this provision could be invoked would be the uplift capacity of a system with spread footing foundations.
- The amplified force demands, as defined by the system overstrength factor, Ω_0, as defined in ASCE 7 (ASCE, 2005).

For SCBF, the connection strength must exceed the lesser of the first two elements in this list, fully ensuring that the connections are not the weak elements in the system.

For SCBF, both the tensile and flexural strength must be considered in the design of the connections. The flexural strength of the connections in the direction of brace buckling is required to be greater than the nominal moment capacity of the brace, unless they are specifically designed to provide the expected inelastic rotations that can be generated in the postbuckling state. This type of detail typically includes a single gusset plate where there is adequate separation between the end of the brace and the connecting element so that the gusset plate can bend unrestrained, as developed from research at the University of Michigan (Astaneh, 1989). In addition, the potential for buckling of gusset plates that may be used in bracing connections must be addressed. Finally, bolted connections should be checked for local failure mechanisms, such as net tension and block shear rupture, to ensure that these potentially brittle modes are avoided.

The eccentrically braced frame (EBF) was systematically developed through years of research at the University of California by Professor Egor Popov and his students, was the first to explicitly require that elements and connections within the system be designed to limit the inelastic response to special members known as "links." For example, in the 2005 AISC *Seismic Provisions*, the design of connections between links and brace elements must consider both the expected overstrength of the material and the strain hardening that is expected to occur in properly detailed link elements. The design of such connections must also be detailed such that the expected response of the link elements is not altered.

In a number of EBF configurations, the link beams are located at the end of a bay, adjacent to a supporting column. Since severe inelastic rotation demands are expected in link beams during major seismic events, there was concern that without special precautions, link-to-column connections in these EBF configurations may be subject to the same type of connection fractures that numerous moment connections suffered in the Northridge earthquake. As a result, the provisions require that these connections be tested to demonstrate that they have adequate rotation capacity. Without testing to qualify the connection detail, the links are conditions that are required to be proportioned to yield in shear and the connections must be reinforced to preclude inelastic behavior at the face of the column.

Two new systems were introduced in the 2005 AISC *Seismic Provisions*. The first of these is the buckling restrained braced frame (BRBF). This special class of concentrically braced frame relies on brace elements that are specially designed to preclude compression buckling over the length of the member. As a result, the energy dissipation and ductility of these braces is significantly improved over that of conventional braced frames. In BRBF's, the tension and compression capacity of the braces are approximately equal, with the compression capacity being approximately 10% greater in most cases. As with the other systems, the connections between the braces to the other members of the frame are designed for the expected capacity of the braces, increased by 1.1 to account for potential strain hardening.

The other system introduced in the 2005 AISC *Seismic Provisions* is the special plate shear wall (SPSW). In this system, thin steel plates are connected to horizontal and vertical boundary elements.

(HBE and VBE). The plate elements are designed to yield and behave in a ductile manner. The connections between the plates and the boundary elements are designed to develop the expected capacity of the plate. In addition, the connections between the HBE and VBE are required to be fully restrained moment resisting connections designed to meet the requirements for OMFs. In addition the shear capacity of this connection must be able to transfer the vertical shear induced by the yielded wall plates.

5.3 Design of Special Moment-Frame Connections

5.3.1 Introduction

This section provides an overview of the requirements and concepts for the design of special moment frame (SMF) connections. The design basis presented is established based on the recommendations of the SAC Joint Venture "Guidelines, FEMA 350" (FEMA, 2000a, 2000b) as well as requirements given in the AISC *Seismic Provisions* (2005), and AISC 358 *Prequalified Connections for Special and Intermediate Steel Moment frames for Seismic Applications* (AISC, 2005). First, general concepts and objectives for design will be outlined, followed by specific connection types and design examples.

Figure 5.1 shows a typical unreinforced detail for a beam-to-column connection. The beam-to-column connection must be capable

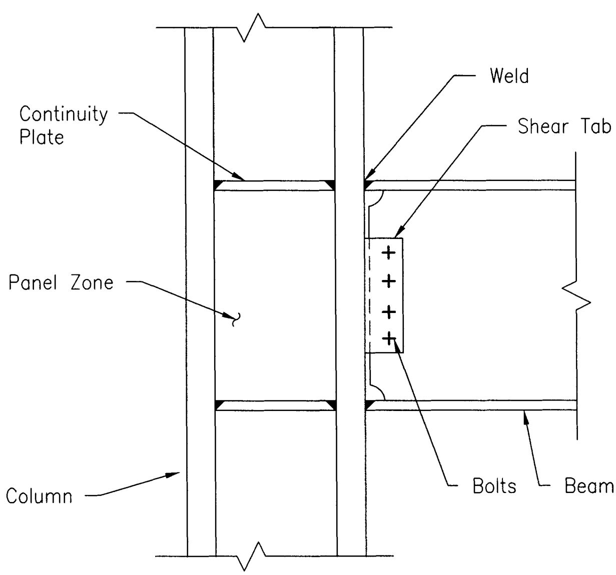


Figure 5.1 One-sided moment frame connection.

of transferring both the beam shear and moment to the column. Historically, the assumption for design was that the beam shear is transferred to the column by the beam web connection and the moment is transferred through the beam flanges. Numerous studies after the 1994 Northridge by FEMA and others demonstrated that this assumption is far different from the actual behavior. Common practice prior to the Northridge earthquake was to either bolt or weld the web to the column shear plate and to weld the beam flanges to the column flange using a full-penetration groove weld. The panel zone (the column web at the beam intersection) is subjected to a shear force due to these moments applied by the beam.

In the design of SMF connections the engineer must set objectives for both load and deformation capacities. Specifically, the load capacity requirement is based on the maximum attainable moment in the beam. The connection to the column must be sufficiently strong to develop the strength of the beam, thus reducing the risk of brittle failure in the connection. Inelastic deformation capacity is required to ensure ductility in predetermined locations under large deformation demands.

Load capacities. A common philosophy adopted since the Northridge earthquake has been to design the connection at the column face to remain nominally elastic, and force the inelastic deformation to occur in the beam itself. The design strength of the connection between beam and column is determined by using a "capacity design" approach. The maximum probable moment and shear that the beam is capable of achieving are determined based on the probable strength of the beam. These maximums then become the design loads for the connection. The connection to the column is then designed based on nominal material properties.

The ability to estimate the maximum moment developed in the beam becomes quite important given the uncertainties regarding actual material behavior. The connection should be designed with the expectation of both beam overstrength and strain hardening in the plastic hinge region. A methodology for estimating the probable moment in the plastic hinge was presented in FEMA 350 (FEMA, 2000a). The approach taken here is based on the FEMA 350 methodology. A similar approach is presented in the AISC *Seismic Provisions* (2005).

Beam overstrength should be accounted for by using the expected yield strength of the beam material. For example, the expected strength of A992, grade 50 steel is approximately 55 ksi, based on mill certificate test values. So for A992, grade 50 steel, the expected yield stress increase from the nominal is (55/50) = 1.1. (*Note:* For ASTM A36 steel this value is 1.5.) This factor is known as R_y in the AISC Seismic Provisions.

The strain-hardening effect in the beam can be quantified by applying a factor of 1.1 to the expected flange yield stress. Recent connection testing (Yu et al., 1997) has shown that an increase by a factor of 1.1 or higher is reasonable to account for strain hardening of the beam in the plastic hinge region. The resulting increase, is known as C_{pr} in AISC 358.

The location of the plastic hinge also must be accounted for. If the plastic hinge occurs at the face of the column ($x = 0$ in), the moment at the column face, M_f, will equal M_{pr}. However, it has been shown by numerous tests that the plastic hinge in a conventional SMF connection typically occurs away from the column face (or end of strengthened beam section), at a distance of approximately $x = d/3$ to $d/4$. Extrapolation over this distance to the column face results in an increased moment demand at the face of the column.

The moment demand at the column face is determined as follows (see Fig. 5.2):

1. Determine the maximum probable plastic moment of the beam, M_{pr}, including overstrength and strain hardening:

$$M_{pr} = C_{pr}R_yM_p = C_{pr}R_yZ_bF_y \tag{5.1}$$

where $C_{pr} = 1.2$ for A992 per AISC 358 (AISC, 2005).

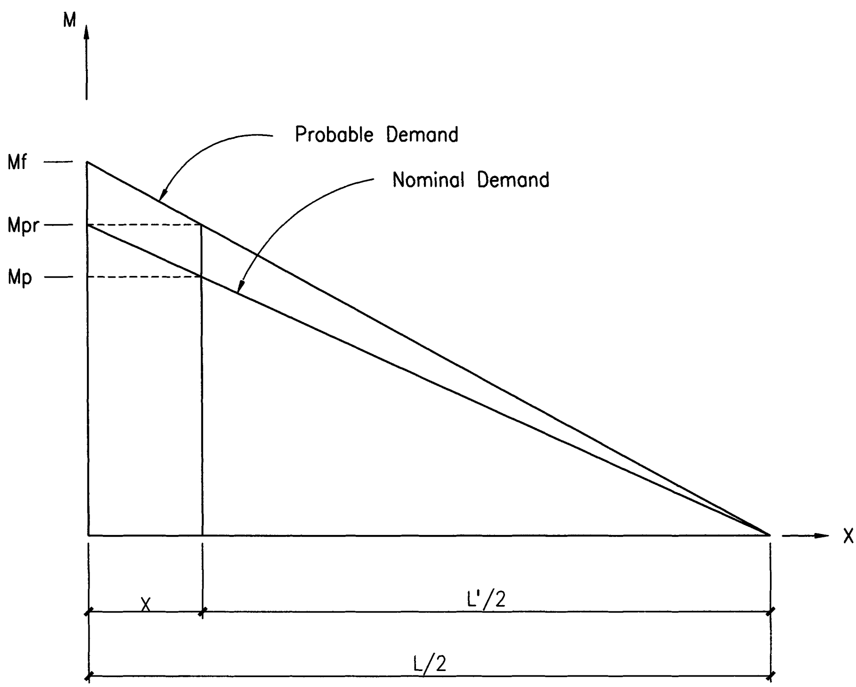


Figure 5.2 Beam seismic moment diagram.

2. Extrapolate the moment to the column face from the assumed seismic inflection point at beam midspan to find the maximum beam moment, M_f:

$$M_f = M_{pr} + V_p x \tag{5.2}$$

where the shear

$$V_p = \frac{2\,M_{pr}}{L'} + \text{factored gravity loads at the hinge location}$$

$$M_f = M_{pr}\left(1 + \frac{2x}{L'} + \right)\text{factored gravity moment at column face} \tag{5.3}$$

3. The shear demand at the column face will be

$$V_f = \frac{2M}{L} + \text{factored gravity loads at the column face} \tag{5.4}$$

Thus, the nominal capacity of the connection at the column face must be designed to resist the load demands M_f and V_f.

Deformation capacities. Obtaining large story drift ratios in an SMF is dependent on the inelastic rotation capacity of the connections. This inelastic rotation may occur by hinging of the beam or column, or by shear yielding in the panel zone, or by a combination of these effects. As the strong column–weak beam (SCWB) is commonly preferred to weak column–strong beam (WCSB), the case of column hinging is not covered here. See Roeder et al. (1990) for further information on WCSB performance.

The story drift of a moment frame is closely related to the total joint rotation. This rotation is composed of both the elastic and inelastic deformations in the frame members (plastic hinges in the beam, shear yielding in the panel zone, etc.). Inelastic deformations in each component of the connection add cumulatively to the total plastic rotation of the connection. This parameter has become a valuable tool in determining the acceptability of connection designs. Connections that have exhibited adequate plastic rotation capacity in tests are generally expected to perform better in seismic events. Inelastic rotation demands may be estimated during the design using various nonlinear analysis techniques.

For special moment frame systems, AISC requires a minimum level of approximately 0.03 rad of plastic rotation (corresponding to the 0.04 rad drift angle) in a qualifying test that follows a specified loading protocol. This may be obtained by a combination of yielding in the beam, panel zone, or column. The ability of a connection to withstand such deformation without significant loss of strength is heavily dependent on ductile detailing of the entire connection region.

5.3.2 Post-Northridge developments in connection design

Historically, moment connection design has relied on the previously described load-transfer assumptions and welded flange/bolted web connection details to allow the strength of the beam to fully develop prior to connection failure. Tests performed by Popov and Stephen (1970) and others indicated that this type of detail could be used for design, as the beam plastic moments were reached and, in some cases, significant amounts of ductility were observed. Indeed, it was believed that the typical steel SMF was well equipped to withstand large seismic force and deformation demands.

With the connection fractures caused by the Northridge earthquake came new questions related to this force transfer mechanism. Was the pre-Northridge connection detail fundamentally flawed? Can it be substantially improved by proper control over material and workmanship? Soon after the Northridge connection fractures were discovered, practitioners and researchers alike began to investigate these questions, and ultimately, to arrive at connection details that can be relied upon to deliver sufficient levels of force and deformation capacity.

Many successful testing programs were performed that now provide guidance and direction for SMF connection design to engineers. Full-scale testing has become an extremely useful tool in helping to understand SMF connection behavior.

For SMF and IMF AISC 341 requires the use of connection designs which have been proven to consistently perform well in tests. Due to the variation of member sizes, material strengths, and other variables between projects, project-specific testing programs may be needed. Alternatively, AISC provides specific acceptance criteria for using past test results of comparable connection designs (AISC, 2005), or AISC's 358 Prequalification Standard, (AISC, 2005).

One of three primary philosophies; (1) a toughening scheme, (2) a strengthening scheme, and (3) a weakening scheme have been used in the development of post-Northridge connection concepts. Often, some or all of these schemes are used in combination.

5.3.3 Toughened connections

Design philosophy. To toughen the connection, significant attention is paid to the complete-penetration weld details between the beam and the column. Notch-tough electrodes are now typically specified (a common requirement complete joint penetration beam flange to column flange welds is for Charpy V-notch values of 20 ft-lb at −20°F and 40 ft-lb at 70°F). In addition, bottom flange backing bars are removed and replaced with reinforcing fillet welds in order to eliminate the notch effect at the root pass of the weld and to remove any weld flaws, which are more prevalent at the bottom flange where the beam web prevents continuous weld passes. At the top flange it is not common to remove the backing bar simply to add a reinforcing fillet to secure the bar to the column flange. Research performed by Xue et al. (1996) supports this approach.

This scheme may be used either as a stand-alone design method or as a supplement to either of the second two schemes. The use of a notch-tough electrode and corrective measures for the backing bar notch effect are critical components to any connection design. In short, taking such measures to toughen the groove weld is considered as a minimum amount of effort to ensure adequate ductile behavior of the connection, but likely do not fully meet the SMF rotation requirements. Other recommendations include improved control in welding and inspection practices.

Both the FEMA/SAC project and AISC studied the unreinforced connection in depth. Two key issues that were studied were the beam web connection and the configuration and preparation of the weld access holes adjacent to the beam flange welds. It was determined that in order to achieve SMF level inelastic rotation demands, the beam web to column connection should be a complete joint penetration weld. In addition the weld access hole preparation should take on a certain shape and size depending on the thicknesses of the beam flange and web. This configuration is depicted in the AISC *Seismic Provisions*. IMF performance can be achieved with a bolted web connection and the improved access hole configurations. These details were first provided in FEMA 350 (FEMA, 2000a), and are being developed as prequalified connections by AISC for publication in the next edition of AISC 358 in 2010.

Another important aspect of the connection is the addition of column continuity plates. Although the use of continuity plates has been based on member geometry for some time, it is now recommended that unless otherwise justified by testing, "continuity plates be provided and that the thickness be at least equal to the thickness of the beam flange for two-sided connections." Welding of continuity plates to column flanges should be performed with full-penetration groove welds, while the plate-to-column web weld may be a double-sided fillet. Notch-tough electrodes should be used in all cases, and care should be taken to avoid welding in the k region of the column.

5.3.4 Strengthened connections

Design philosophy. Another method of ensuring sufficient connection capacity is by strengthening the portion of the beam directly adjacent to the column, where the maximum moment occurs during seismic loading. The increased capacity near the column flange, M_f, forces the plastic hinge to form in the unstrengthened section of the beam (see Fig. 5.3).

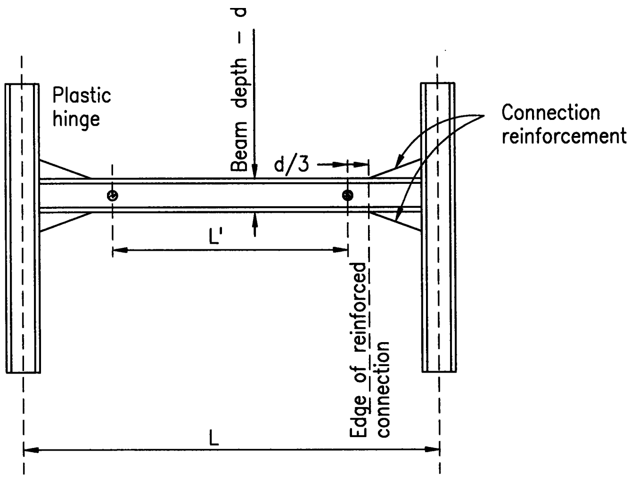


Figure 5.3 Location of plastic hinge in a one-bay frame.

The method used in this approach is to protect the previously vulnerable beam-flange complete-penetration welds with the addition of cover plates, rib plates, side plates, or haunches at the beam-to-column interface. The effective section modulus of the beam at the connection is increased, which decreases the bending stress at the extreme fiber of the section, as well as the total force resisted by the flange welds.

Strengthening these connections will invariably increase the stiffness of the frame. The effect this has on determining story drifts and building period must be considered in the design, but in most cases is relatively minor.

Another consideration is the satisfaction of the AISC requirements for panel zone strength and the strong column–weak beam condition. The extrapolated moment, M_f, can now be well above the beam plastic moment, M_p, and must be considered. The AISC *Seismic Provisions require* minimum level of panel zone strength so that the panel zone can share the inelastic response with the beam hinges.

Cover-plated connections. In the years immediately following the Northridge earthquake one popular method of strengthening the connection was to weld cover plates to the top and bottom beam flanges. Full-scale testing of cover-plated connections in was performed by Engelhardt and Sabol (1995), Noel and Uang (1996), and others. In general, these tests showed the ability of cover-plated connections to perform well in the inelastic range, and it was included in FEMA 350 (FEMA, 2000a).

Proper detailing is essential to obtain ductile behavior from a cover-plated connection. Typically, cover plates are fillet-welded to the beam flanges and groove-welded to the column flange. A common detail is

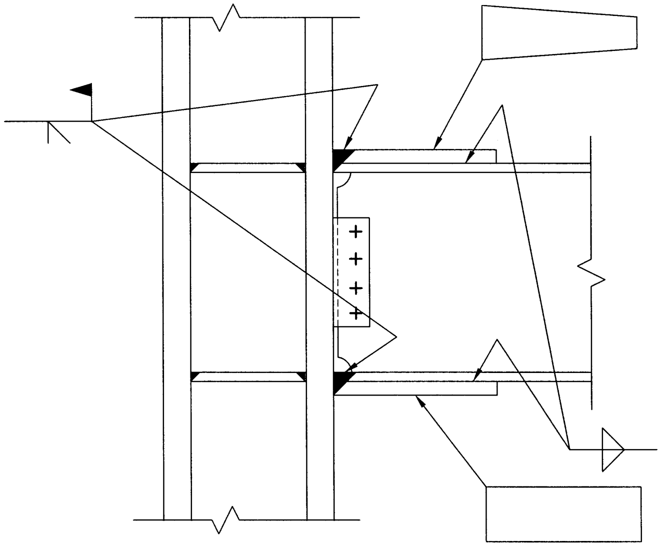


Figure 5.4 Cover-plate detail.

shown in Fig. 5.4. For ease of field erection, the bottom cover plate is oversized and the top plate undersized, to allow for downhand welding at each location. A variation to this technique uses oversized top and bottom cover plates, with the top plate shop-welded to the beam and the bottom plate field-welded. This allows the use of wider plates, while allowing downhand welding at both locations.

Note that only the long sides of the cover plates are welded to the beam flange. Welds loaded in the direction of their longitudinal axes perform significantly better in the inelastic region than those loaded in a perpendicular direction (AISC, 2005), hence cross-welds to the beam flanges at the end of the cover plates are not recommended.

Another detailing issue is the type of groove weld used at the cover-plate–to–column-flange connection. Two options are shown in Fig. 5.5 for this weld detail. Type I is the preferred detail. Although the type II detail uses less weld metal, the sharp angle of intersection between the cover-plate weld and the beam-flange weld creates a less desirable "notch" effect. From a fracture mechanics standpoint, the type II detail is more susceptible to horizontal crack propagation into the column flange. The designer must consider the amount of heat input and residual stresses in the joint region for either type detail. It is good practice to have a maximum total weld thickness of 2 times the beam flange thickness, or the thickness of the column flange, whichever is less. This is a means to conserve the amount of heat input to the welded joint region.

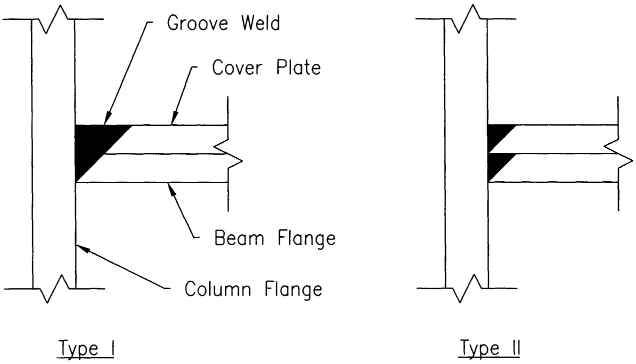


Figure 5.5 Cover-plate groove-weld types.

The thickness of the cover plate used is an essential variable to consider. The area of weld required between the strengthened beam section and the column face must be sufficient to resist the amplified beam moment, M_f. Once the required cross-sectional area of weld is obtained, it may be comprised of a combination of beam-flange weld and cover-plate weld or by plate weld alone if a thicker plate is used. The latter, known as a flange-plate connection, provides no direct connection of the beam flanges to the column flange, only to the cover plates. Full-scale tests of this type of connection were reported by Noel and Uang (1996) (see Fig. 5.6). If this option is chosen, care must

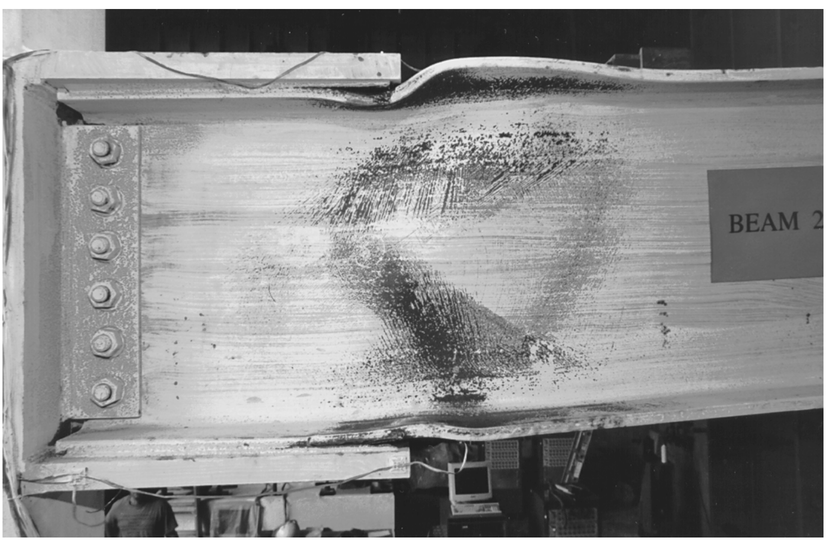


Figure 5.6 Cyclic performance of a flange-plated connection. (*Courtesy of Forell/Elsesser Engineers, Inc., San Francisco, CA.*)

be taken to avoid deformation incompatibility between the thin beam flange and relatively thick cover plate, resulting in premature fracture of the longitudinal cover-plate fillet welds.

Haunched connections. Another method of strengthening the connection is by the addition of a haunch at the beam-flange–to–column-flange connection. The haunch is typically located on the bottom flange only, due to the presence of the floor slab on the top flange. The addition of a haunch to both flanges is a more expensive option, but has been shown to perform extremely well in tests. Haunches are typically made from triangular portions of structural tee sections or built-up plate, and stiffeners are provided in the column and beam webs (see Fig. 5.7).

Full-scale testing of bottom flange–welded haunch connections to date includes work done by Engelhardt et al. (1996), Popov and Stephen (1970), Uang and Bondad (1996), and Noel and Uang (1996). Whittaker, et al. (1995) report good performance of connections made with top and bottom flange–welded haunches. The bolted haunch was studied by Ksai and Bleiman (1996). These details are also included in FEMA 350 (FEMA, 2000a).

Although work by Yu et al. (1997) questioned the validity of the classical beam theory bending stress ($f = Mc/I$) in haunch design, a number of test specimens designed using this theory have performed very well (see Fig. 5.8). The geometry of the haunch should be such that: (1) the

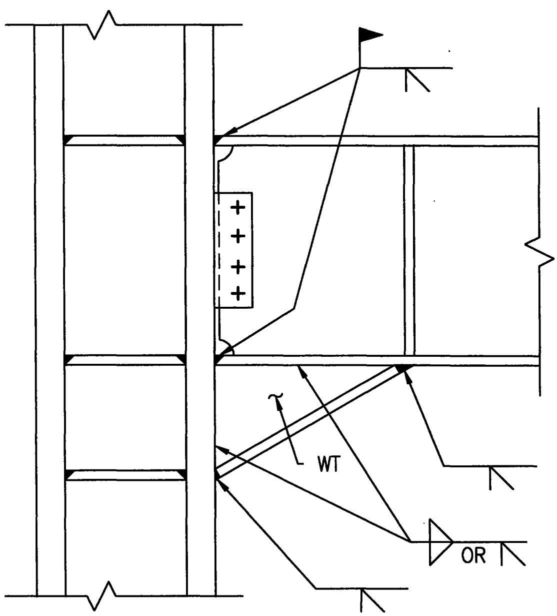


Figure 5.7 Bottom flange haunch connection.

Figure 5.8 Yielding and buckling patterns of a beam subjected to cyclic loading. This connection incorporates both a bottom flange haunch and a top flange cover plate. (*Courtesy of Forell/Elsesser Engineers, Inc., San Francisco, CA.*)

moment, M_f, is resisted while satisfying the $0.9F_{yc}$ through thickness requirement and (2) the haunch aspect ratio is sufficient to develop adequate force transfer from the beam flange. A moderate balance is required here as the longer the haunch is, the higher the demand moment at the column face, M_f, becomes. The design methodology presented by Yu et al. (1997) recognizes a more realistic force transfer mechanism in the haunch connection. In this approach, the haunch flange is modeled as a strut which attracts vertical beam shear, hence reducing the beam moment, M_f, and the tensile stress at the beam flange welds.

Vertical rib-plate connections. A vertical rib plate serves a similar purpose as the cover plate and the haunch; strengthening the section by increasing the section modulus while distributing the beam-flange force over a larger area of the column flange (see Fig. 5.11). The engineer may place a single rib plate at the center of the beam flange, but a more common practice is to position multiple ribs on each flange to direct the beam-flange force away from the center of the beam flange. By doing so, the stress concentration at the center of the beam-flange groove weld is somewhat alleviated.

Testing of rib-reinforced connections was been limited, but a few examples have shown that this method of strengthening can lead to ductile connection behavior (Engelhardt and Sabol, 1995; Anderson, 1995; and Tsai and Popov, 1988).

It should be noted that while meeting the intent of providing substantial inelastic rotation performance can be met by the various strengthened connection approaches, they have not been widely used because of the extra fabrication and erection expense when compared to other details.

5.3.5 Weakened connections

Design philosophy. Weakening the connection is achieved by removing a portion of the beam flange to create a reduced beam section, or RBS (see Fig. 5.9). The concept allows the designer to "force" a plastic hinge to occur in a specified location by creating a weak link, or fuse, in the moment capacity of the beam. Figure 5.10 shows the moment diagram of a beam under seismic loading. The geometry of the RBS must be such that the factored nominal moment capacity is not exceeded, at the critical beam section adjacent to the column.

This method has potential benefits where the strengthening scheme had drawbacks. With a reduced M_p, the overall demand at the column face, M_f, must, by design, be less than the nominal plastic moment of the beam. Therefore, SCWB and panel zone strength requirements are easier to achieve.

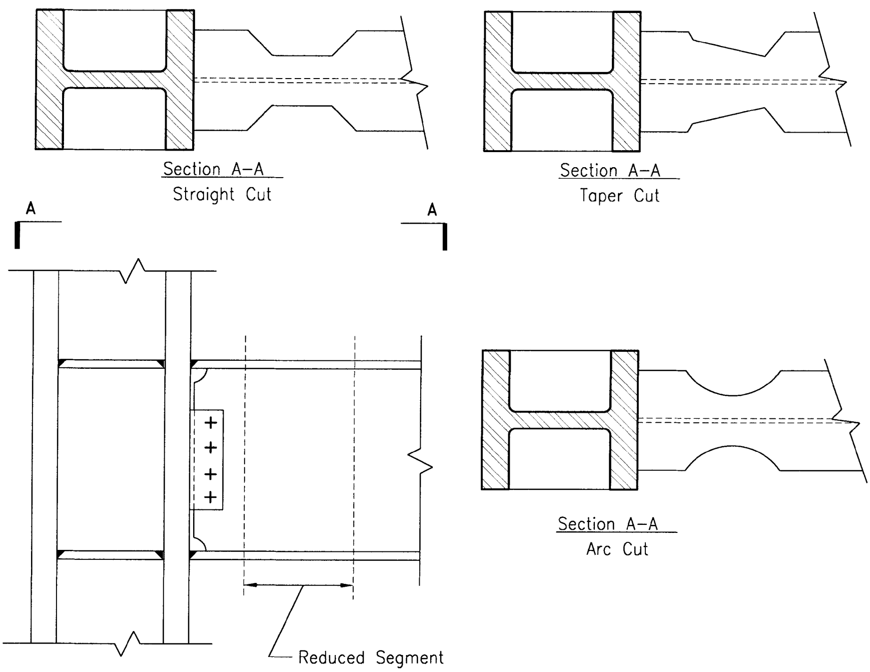
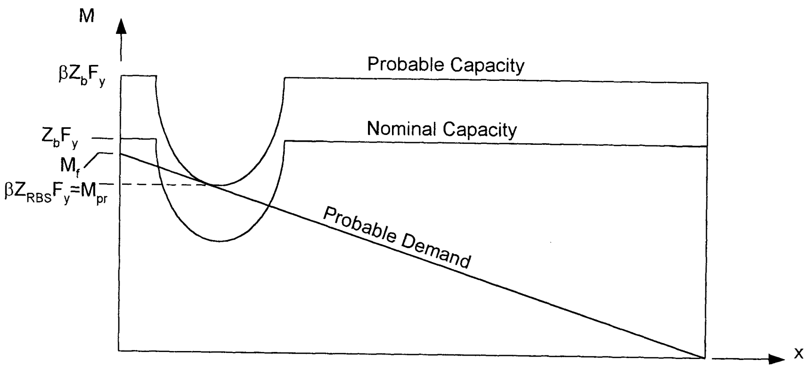


Figure 5.9 Reduced beam section connection.



(a) RBS Seismic Moment Diagram

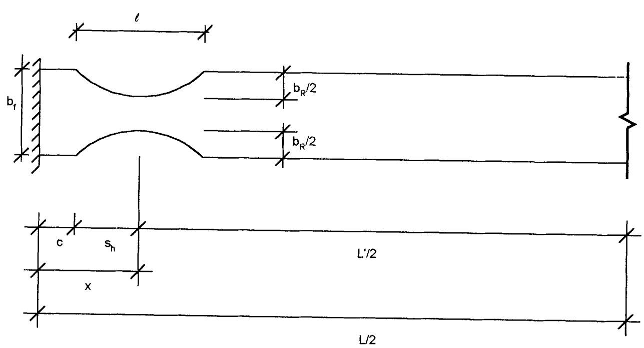


Figure 5.10 Reduced beam section moment diagram and flange geometry: (*a*) RBS seismic moment diagram and (*b*) RBS beam flange geometry (arc-cut section).

The drawbacks for RBS come in the form of reduced stiffness. The reduction in overall lateral frame stiffness is typically quite small (typically on the order of a 5% to 7% reduction). On the other hand, the reduction in the flange area can significantly reduce the stiffness (and stability) of the beam flange, creating a greater propensity for lateral torsional buckling of the beam in the reduced section. The addition of lateral bracing is recommended for Lateral bracing near the RBS may be required if a structural slab is not present or if above minimum acceptable performance is desired.

Choice of RBS shape. The shape, size, and location of the RBS all can significantly affect the connection performance. In the early studies various shapes were tested, as noted schematically in Fig. 5.9. Test programs were performed to investigate straight cut (Engelhardt et al., 1996), taper cut (Iwankiw and Carter, 1996; Uang and Noel, 1996), arc cut (Engelhardt et al., 1996) and drilled flanges.

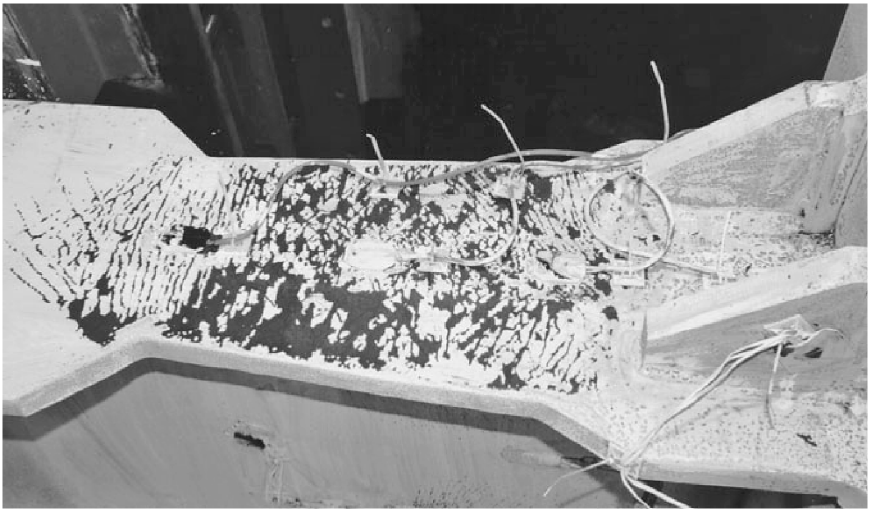


Figure 5.11 Yielding in the reduced section of a "taper-cut" beam flange subjected to cyclic loading. (*Courtesy of Ove Arup and Partners, Los Angeles, CA.*)

Each RBS shape has benefits and shortcomings relative to each other. For instance, tapered cuts allow the section modulus of the beam to match the seismic moment gradient in the reduced region. This creates a reliable, uniform hinging location. However, stress concentrations at the reentrant corners of the flange cut may lead to undesired fracture at these locations as reported by Uang and Noel (1996) (see Fig. 5.11). Curved flange cuts avoid this problem, but do not give the benefit of uniform flange yielding, although test results indicate that plastification does distribute over the length of the reduced section.

The lack of sharp reentrant corners and the ease of cutting made the circular arc-cut reduction a preferred option. In general, tests performed on arc-cut RBS connections have provided favorable results (Engelhardt, et. al., 1996). The design methodology presented by FEMA 350 and AISC 358 is applicable to curved arc reduction cuts.

Geometry determination. Once a suitable bean size for frame drift is obtained, sizing the cut becomes the next obstacle. Keeping in mind the requirements for connection capacity at the face of the column (see Sec. 5.3.1), as well as gravity loading demands at the location of the RBS, the size of reduction must be chosen appropriately.

Since member sizes in SMFs are typically governed by drift requirements, it is initially assumed that the reduced section will still work for strength under gravity loading. This load case must be checked after the geometry is chosen based on seismic loading.

Given a beam span L, depth d, and hinge location s_h, the reduction variables l, c, and b_R (see Fig. 5.10) define the seismic moment gradient and may be tailored to satisfy the requirements described previously. The majority of RBS connection tests have used relatively similar values for these essential variables. For instance, the length of reduction, l, has typically ranged between $0.75d$ and d. The distance of the RBS away from the column face, c, has typically been chosen as approximately $0.25d$, however, work by Engelhardt, et al. (1996) justifies using a value of $0.75\ b_f$. These values were shown to be effective in a number of testing programs.

The width of flange which is removed, b_R, determines the plastic modulus at the reduced section, $Z_{\mathrm{RBS}} = Z_x - b_R t_f (d - t_f)$. This reduced modulus is then used to calculate the moment at the column face, M_f, using the method shown in Sec. 5.3.1. A practical upper bound on the value, b_R, has generally been 50% of the flange width, b_f. This limit is based on both stability and strength considerations. Excessive reduction can lead to premature lateral torsional buckling of the beam, which should be avoided. In the event that even a 50% flange reduction does not sufficiently reduce M_f. Supplemental strengthening may be considered in the area between the RBS and the column face. Reinforcing ribs at the column face have been shown to enhance the performance of RBS connections in tests (Uang and Noel, 1996).

Example 5.1 RBS Connection Design. Design an RBS connection between a U36 × 150 beam and a W14 × 426 column. The beam span in 30 ft. The flange reduction will be an arc-cut shape. We will use the guidelines of AISC 358 and gravity loads will be neglected (see Fig. 5.12).

$$L' = L - d_c - 2x = 302 \text{ in}$$

$$M_{pr} = \mathrm{C_{pr} R_y} Z_{\mathrm{RBS}} F_y = 1.2(Z_{\mathrm{RBS}})(50 \text{ ksi})$$

$$= 60 Z_{\mathrm{RBS}}$$

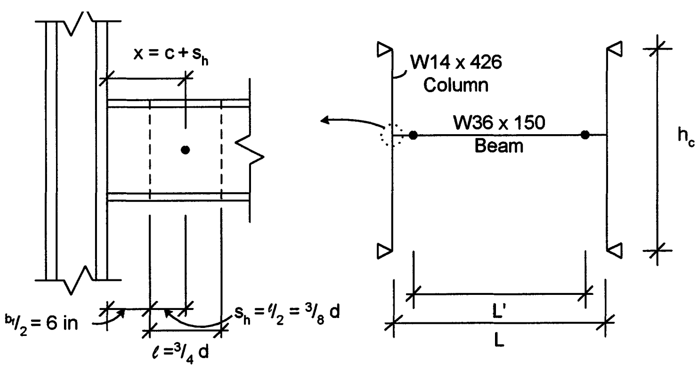


Figure 5.12 Frame and connection used in Example 5.1.

$$V_p = \frac{2M_{pr}}{L'} = \frac{2(60\text{ ksi})Z_{\text{RBS}}}{302\text{ in}}$$

$$= 0.39Z_{\text{RBS}}$$

$$M_f = M_{pr} + V_p x = Z_{\text{RBS}}[60 + 0.39(19.5\text{ in})]$$

$$= 67.6\, Z_{\text{RBS}}$$

- Find the required flange reduction:

$$Z_{\text{req}} = \frac{M_f}{50\text{ ksi}} = 1.35\, Z_{\text{RBS}} \qquad Z_{\text{req}} = Z_b$$

$$Z_{\text{RBS}} \leq 0.74\, Z_b = 430\text{ in}^3$$

$$Z_{\text{RBS}} = Z_x - b_R t_f(d - t_f)$$

$$430\text{ in}^3 = 581\text{ in}^3 - b_R(1\text{ in})(35.85\text{ in} - 1\text{ in})$$

$$b_R \geq 4.3\text{ in}$$

$$\frac{b_R}{b_f} = \frac{6\text{ in}}{12\text{ in}} = 50\%\text{ reduction, ok}$$

∴ Try a 6-in flange reduction

$$Z_{\text{RBS}} = 372\text{ in}^3$$

$$M_f = 67.6(372) = 25{,}147\text{ kip-in}$$

- Check the through-thickness stress:

$$f_{tt} = \frac{M_f}{S_b} = \frac{25{,}147}{504} = 49\text{ ksi} = \phi F_{ye} = 0.9(54\text{ ksi}),\text{ ok}$$

∴ Use a 6-in beam-flange reduction (50%) (see Fig. 5.13).

- Check that panel zone strength and SCWB requirements will meet the requirements of the AISC *Seismic Provisions*.
- Continuity plates:

Add 1-in-thick continuity plates at the top and bottom flange level, to match the beam flange assuming a two-sided frame configuration.

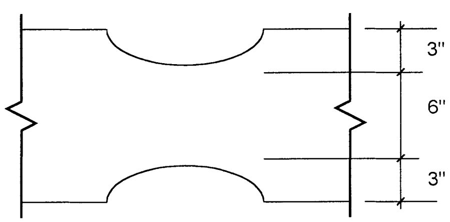


Figure 5.13 RBS flange reduction.

It should be noted that the preceding discussion presents some, but not all, of the connection design approaches that have been developed since the Northridge earthquake. In fact, a few approaches were patented and have been widely used; these patented connections have not been addressed here.

5.4 Concentrically Braced Frames

5.4.1 Introduction

Concentric braced frames have found wide application in lateral force-resisting systems, typically having been chosen for their high elastic stiffness. This system is characterized by horizontal and vertical framing elements interconnected by diagonal brace members with axes that intersect. The primary lateral resistance is developed by internal axial forces in the framing members. The AISC *Seismic Provisions* make a distinction between ordinary concentrically braced frames (OCBF) and special concentrically braced frames (SCBF). SCBF frames are specifically detailed and typically sized to withstand the full inelastic behavior of the lateral system. This section will describe the connection design for both types of concentric braced frames.

Figure 5.14 shows several types of braced frames. The V-braced systems shown require the intersected beams to be specially designed when used in SCBF structures in order to ensure their stability once the bracing system begins to exhibit inelastic behavior. During a large earthquake, it is expected that the compression brace will buckle before the tension brace begins to yield. At the connection to the beam, there is an imbalance of forces from the braces that needs to be resolved by the beam member. As the lateral loading continues and both braces yield, the maximum force imparted to the beam will be the difference in the strengths of the buckling brace and the tension yielding brace. The direction in which this force acts depends on the bracing configuration. The brace connections and the beam need to be able to transfer these loads.

SCBF systems have different requirements than OCBF systems. The width-thickness ratio of the rectangular tube sections in SCBFs is limited to $\sqrt{0.64\ E/F_y}(F_y)$. This is intended to minimize local buckling of the brace elements and results in larger wall or flange thickness. Since the connections are typically designed for the brace capacity in SCBF systems, the force level for the design of the connection will increase.

Due to the better behavior of the system, AISC allows more slender elements in SCBFs than in OCBFs. The slenderness of OCBFs are limited to $KL/r \leq 4\sqrt{E/F_y}$, whereas SCBF braces are only limited to

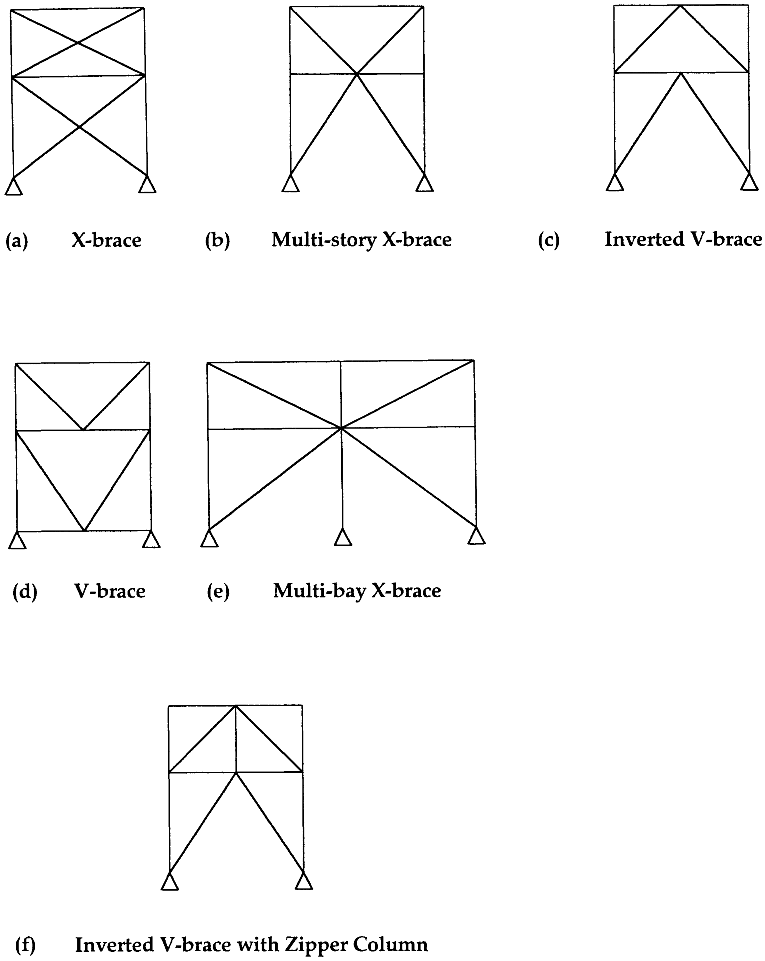


Figure 5.14 Concentric braced frame types: (*a*) X brace; (*b*) multistory X brace; (*c*) inverted V brace; (*d*) V brace; (*e*) multibay X brace; and (*f*) inverted V brace with zipper column.

$KL/r \leq 4\sqrt{E/F_y}$. This seems to contradict testing which has shown that the hysterectic response during inelastic cyclic reversals improves as the slenderness of the compression member decreases. Locally, brace behavior is improved with stocky members, however, inelastic analyses which analyze the entire system indicate that large reductions in the slenderness can cause the compression capacity to approach the tension capacity, which results in a soft story effect. This will occur if, once the compression braces of a story buckle, the tension members on the same story yield before compression members on other floors buckle. Since the buckling strength is close to the tension capacity, the postbuckling reduction in strength is often enough to yield the adjacent tension members. The addition of a "zipper column" as shown in Fig. 5.14, avoids this condition by better distributing the forces throughout the height of the system as the members exceed their elastic limits.

5.4.2 Connection design and example

This section will present an example design of a connection in a special concentrically braced frame. Throughout the example, it will be noted how the criteria would differ for an ordinary concentrically braced frame. Figure 5.15 shows the brace configuration and Example 5.2 presents a spreadsheet outlining the entire connection design. The connection presented addresses the intersection of a tube steel brace with a beam-column connection. Similar approaches may be followed for other brace configurations and section types.

Force level. The design of the connection is dependent on the design forces during compression and tension. Designing the connection for the capacity of the member ensures the connection is not the yielding element in the system. The maximum force the connection will be subject to is the yielding of the brace member in tension defined as $R_y F_y A_g$.

The R_y factor accounts for the expected material overstrength and strain hardening of the member. Had this connection been designed for use in an OCBF, the design force level could have been reduced to the maximum expected force as defined by the maximum force that can be delivered by the system, or a load based on the Amplified Seismic Load in ASCE 7 (ASCE, 2005).

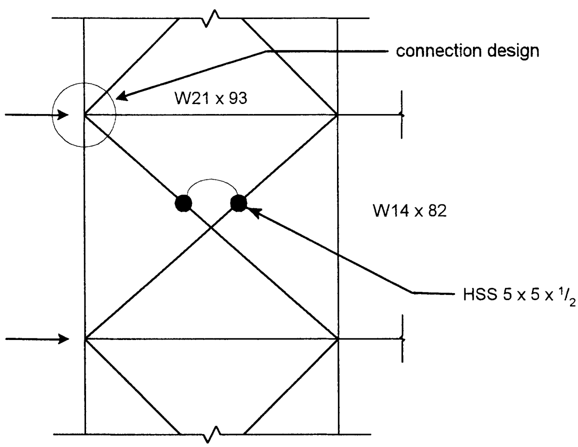


Figure 5.15 Design example.

The spreadsheet analysis begins by determining the section sizes, material, and geometry which will determine the brace's force magnitude and direction at the connection.

Force distribution. The force distribution from the brace to the beam and column can be calculated using the uniform force method described in AISC publications. This method provides a rational procedure for determining the interface forces between the gusset plate and the horizontal and vertical elements at the connection. The axial and shear forces are distributed in the connection based on stiffness, while the required moment for equilibrium is assigned to the beam or column or to the beam and column equally.

An alternate method to determine the forces in the connection may use the fin truss approach originally proposed by Whitmore and modified by Astaneh (1989). This approach discretizes the gusset plate into radial elements and distributes the force based on axial stiffness and the angle of incidence. The procedure has been applied successfully on single-member connections, but appears overly conservative for multimember gusset-plate configurations when the forces are not independent of one another.

Example 5.2 defines the geometry of a rectangular plate where the $2t$ offset required to allow an unrestrained bending zone of the brace is provided between the points of support of the plate to the beam and column and the end of the brace. This configuration not only provides a simple plate geometry, but also eliminates the need for stiffeners on the plate. Had the plate utilized smaller leg dimensions, a tapered plate would be required, but the buckling line perpendicular to the brace would start from the bottom edge of the plate upward to a free edge of the plate. This free edge should be supported by stiffener plates to ensure that during buckling the plate remains stable and bends perpendicular to the brace. Should the buckling line migrate to the stiff supported points not perpendicular to the brace, such as the ends of the tapered plate without stiffeners, it is feared that the brace may effectively buckle about a different axis at each end imparting torsional forces into the brace. Figure 5.16 shows this condition.

Example 5.2 shows the dimensions calculated based on the geometry specified and the resulting load distribution using the uniform force method. Figures 5.17 and 5.18 show the geometric variables. The axial force on the beam and the shear force on the column can be significant from the gusset plate. In frames where the brace intersects the column from each side or the beam from top and bottom, large demands may overstress the section requiring either the size be increased or the webs be strengthened with doubler plates.

Brace–to–gusset-plate connection. The connection of the tube brace to the gusset plate uses four fillet welds along a slot to fit the gusset

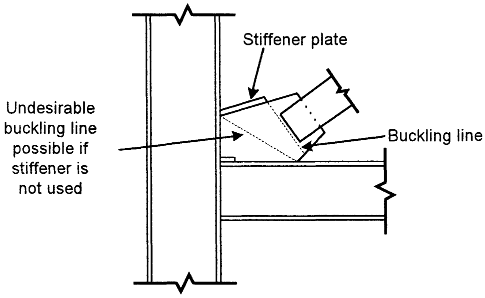


Figure 5.16 Tapered gusset plate with stiffeners.

Example 5.2: Design of Special Concentric Braced Frame Connection

Calculation of special concentric braced frame connection
to accomodate brace buckling behavior and conform to the
AISC Seismic Provisions for Structural Steel Buildings
Uniform Force Method

(INPUT IS INDICATED BY BOLD ITALICS)

Global Input

General

Title: ***TS5x5x1/2 Brace***

Global Data

Location:	(U)pper / (L)ower =	***L***		

Brace Data

Shape:	Shape =	***TS5x5x.5***		
Orientation:	(S)trong / (W)eak	***W***		relative to the out of plane direction
K_ip:	X-X eff. len. fact. =	***1.50***		brace effective length factor in the plane of the frame
K_op:	Y-Y eff. len. fact. =	***1.00***		brace effective length factor out of the plane of the fram
Fybr:	Yield =	***46***	**ksi**	
Ry:	Overstrength =	***1.1***	**kips**	1.5 for A36, 1.3 for A992, 1.1 for other grades
Put:	Ry*Fy*Ag =	**423**	**kips**	
λ:	KL/(r*pi)*sqrt(Fy/E) =	**1.159**		(Out-of-Plane Buckling Governs)
Pcr:	0.658^(LAMBDA^2)*A*FY	219.18		

Upper beam

Shape:	Shape =	***W21X93***	
TOS:	Top of steel =	***10.00***	**ft**
Fyub:	Yield =	***50.00***	**ksi**

Lower beam

Shape:	Shape =	***W21X93***		
TOS:	Top of steel =	***0.00***	**ft**	
Fylb:	Yield =	***50.00***	**ksi**	
X:	Bay width =	***10***	**ft**	Horizontal length between work points

Column

Shape:	Shape =	***W14X82***	
Fycol:	Yield =	***50.00***	**ksi**
Orientation:	(S)trong/(W)eak =	***S***	

Member Properties Summary

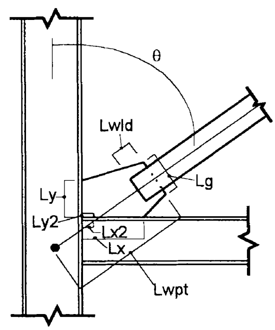
		Brace:				**Upper beam:**	
	section =	TS5x5x.5			section =	W21X93	
Abr:	area =	8.36	in^2	**Abm:**	area =	27.3	in^2
dbr:	depth =	5.0	in	**dbm:**	depth =	21.62	in
bfbr:	flange width =	5.0	in	**bfbm:**	flange width =	8.42	in
twbr:	wall thick. =	0.5	in	**kbm:**	k =	1.6875	in
tfbr:	wall thick. =	0.5	in	**twbm:**	web thick. =	0.58	in
rxx:	X-X rad. of gyr. =	1.8	in (strong axis)	**tfbm:**	flange thick. =	0.93	in
ryy:	Y-Y rad. of gyr. =	1.8	in (weak axis)	**k1bm:**	k1 =	0	in

		Lower beam:				**Column:**	
	section =	W21X93			section =	W14X82	
Abm:	area =	27.3	in^2	**Acol:**	area =	24.1	in^2
dbm:	depth =	21.62	in	**dcol:**	depth =	14.31	in
bfbm:	flange width =	8.42	in	**bfcol:**	flange width =	10.13	in
kbm:	k =	1.6875	in	**kcol:**	k =	1.625	in
twbm:	web thick. =	0.58	in	**twcol:**	web thick. =	0.51	in
tfbm:	flange thick. =	0.93	in	**tfcol:**	flange thick. =	0.855	in
k1bm:	k1 =	0	in	**k1col:**	k1 =	1	in

Connection Geometry

Lwpt:	Length from W.P. to end of gusset along brace =	***42***	**in.**
Ly:	Gusset dim. along col. from face-of-beam =	***22***	**in.**
Ly2:	Face of beam to start of column-gusset weld =	***5***	**in.**
Lx:	Gusset dim. along beam from face-of-column =	***21.07***	**in.**
Lx2:	Face of Column to start of beam-gusset weld =	***5***	**in.**
Lwld:	Weld length for brace flanges-to-gusset pl. =	***12***	**in.**
Lg:	Gusset dimension perpendicular to brace =	***9***	**in.**
t:	Gusset thickness =	***0.75***	**in.**
Fy_gus:	Gusset yield stress =	***36***	**ksi**
Fu_gus:	Gusset Ultimate stress =	***58***	**ksi**
θ:	Theta =	**0.785**	**rad.**
α:	Length to mid-gusset along bm. = (Lx-Lx2)/2+Lx2 =	**13.035**	**in.**
β:	Length to mid-gusset along col. = (Ly-Ly2)/2+Ly2 =	**13.5**	**in.**
eb:	dlb / 2 =	**10.81**	**in.**
ec:	dcol / 2 =	**7.155**	**in.**
L_unb:	Approximate unbraced length of brace =	**9.14214**	**ft.**

Figure 5.17



To specify a rectangular plate

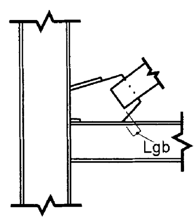
Lx req'd =	25.73	**in.**
Ly req'd =	22.07	**in.**

Note: The brace length is calculated assuming that the same columns are used on each side of the bay and that the basic gusset geometry is the same at both ends of the brace. This length is used to calculate the brace compression capacity.

Bending Zone

t:	Gusset thickness =	0.75	**in.**
2t:	Current value for twice the gusset thickness =	1.5	**in.**
Lgb:	Bending at the beam-to-gusset connection =	2.40	**in. >= 2t OK**
Lgc:	Bending at the column-to-gusset connection =	1.74	**in. >= 2t OK**

Figure 5.18



Compactness, Local Buckling

KL/r: (Out-of-Plane Buckling Governs)	Maximum slenderness ratio =	91.4214	**< 1000/sqrt(Fy) OK**
b/t:	Compactness =	10	**< 110/sqrt(Fy) OK**

Figure 5.17 Gusset-plate connection geometry.
Figure 5.18 Gusset-plate distance requirements.

Moment Reduction I

	Current	Req'd for Zero Mom.	
Lx =	21.07	29.31	**in.**
Ly =	22.00	13.76	**in.**
α =	13.04	17.16	**in.**
β =	13.50	9.38	**in.**

Connection Stiffness

Any moment required for equilibrium will be carried by the stiffer of the beam-to-gusset and the column-to-gusset connection. The designer can choose roughly how to distribute the forces to the framing members.

Distribute forces:

(E)qually to Beam and Column, Primarily to (B)eam, or Primarily to (C)olumn = ***E*** (Enter E, B, or C only)

Moment Reduction II

	Current	Req'd for Zero Mom.		
Lx =	21.07	25.05	**in.**	
Ly =	22.00	17.74	**in.**	
α =	13.04	15.02	**in.**	**Moment on interface**
β =	13.50	11.37	**in.**	**Moment on interface**

Load Distribution Manipulation

The beam and column shear capacities are checked for the additional shear being introduced into them.
Beams may also need to be checked for additional bending stresses.

Load Adjustments				**CONCLUSIONS**
Vbm_r:	Reduce shear in beam by :	***0***	**kips**	
Hcol_2:	Other horizontal shear into column :	***0***	**kips**	
Vbm_2:	Gravity + other seismic shear into beam :	***0***	**kips**	

Final Interface Forces

Gusset-to-Column

Fvcol:	Phi*Aweb*Fv = 0.9x0.6 x fycol x dcol x twcol =	197.049	**kips**	
Hcol:	Horizontal force on column = Ec*Pu/r	96.50	**kips**	
Hcol_Total	Total column shear = Hcol + Hcol_2 =	96.50	**kips**	**OK < Fvcol**
Vcol:	Vertical force on column = Beta*Pu/r	153.32	**kips**	
Mc:	Moment from gusset on column = Huc*(Beta-Beta_bar)	-205.76	**in.-kips**	

Gusset-to-Beam

Fvbm:	Phi*Aweb*Fv = 0.9x0.6 x fybm x dbm x twbm =	338.569	**kips**	
Hbm:	Horizontal force on beam = Alpha*Pu/r	202.62	**kips**	
Vbm:	Vertical force on beam (incl. Vbm_r red.) = Eb*Pu/r - Vbm_r	145.80	**kips**	
Vbm_Total	Total beam shear = Vbm + Vbm_2 =	145.80	**kips**	**OK < Fvbm**
Mbm:	Moment from gusset on beam =	289.82	**in.-kips**	
fbm:	Bending stress on beam = Mb/S	1.51	**ksi**	**OK < Fybm**

Brace-to-Gusset Connection

This section designs the welded connection between the tube walls and the gusset plate. This section applies for TS sections only.
Additional and/or different checks will need to be performed for other shapes.

Brace Flange-to-Gusset Connection

	Electrode =	***E70***		
	Electrode correction =	1	**[LRFD 2nd ed. p8-158]**	
FLFORCE:	Design force in one side of tube = (Pu / 2)=	211.5	**kips**	
efl:	One side eccentricity =	1.88	**in.**	
a:	efl / Lwld =	0.156	**in.**	
c:	Interpolate from weld table values =	2.736	**[LRFD 2nd ed. p8-163]**	
GBlockTear:	Min of 0.75*(0.6*Fy*Agv+Fu*Ant) and 0.75*(0.6*Fu*Anv+Fy*Agt)	454.725	**kips**	**OK > Pu**
Gtear:	Gusset tearout cap. = 0.75x0.6 x Fu x t x Lwld x 2 =	470	kips	**OK > FLFORCE**
TStear:	Tube wall tearout = 0.9x0.6 x Fybr x tfbr x Lwld x 2 =	298	kips	**OK > FLFORCE**
	Required fillet weld size =	6.44	**16ths in**	

Eccentric weld table

a	*c*
0.150	2.750
0.200	2.640

Design of Gusset Plate — **CONCLUSIONS**

Check the tension and buckling capacity of the gusset plate.

Design Data

	Slope (m)	Y intercept y = mx + b
Line A	-1.00	42.43
Line B	3.37	-84.41
Line C	3.73	-89.50
Line D	0.00	32.78
Line E	0.27	23.98
Line F	1.00	3.66

Line Intersect.	Xint	Yint
Lines A & B	29.00	13.43
Lines A & C	27.88	14.55
Lines A & D	9.61	32.82
Lines A & E	14.55	27.88
Lines A & F	19.39	23.04

Tension

Weff:	Effective width of gusset for tension =	18.86	**in.**	
Tcap:	Capacity of gusset in tension = (0.9*Fy x t x Weff) =	458.21	**kips**	**OK > Pu**

Compression

K:	Gusset plate effective length factor =	***0.80***		
	Effective width of gusset for compression =	18.86	**in.**	(per Whitmore's area)
L:	Average Length for Compression =	17.30	**in.**	(used to calculate gusset
ry:	Radius of gyration (weak way) =	0.22	**in.**	comp. capacity)
λ:	Lambda (gusset) = KL/r*pi*sqrt(FY/E)	0.72		
Fcr:	Fcr for gusset =	29.03	**ksi**	
Pcap:	Compression capacity of gusset = 0.85*Ag*Fcr =	349.02	**kips**	**OK > Pbuckle**

Design of Gusset-to-Beam Connection

Design of the gusset-to-beam coonection as a welded connection.

Gusset-to-beam interface forces

Hbm:	Horiz. force at the gusset-beam interface = Hb	202.62	**kips**
Vbm:	Vertical force at the gusset-beam interface = Vb	145.80	**kips**
Mbm:	Moment at the gusset-beam interface = Mb	289.82	**in.-kips**

Gusset Stresses

Fvcap:	Gusset shear capacity =0.9x0.6 x Fy =	19.44	**ksi**	
fvult:	Shear stress on gusset = Hbm / (t x (Lx-Lx2)) =	16.81	**ksi**	**OK < Fvcap**
fault:	Axial stress = Vbm / (t x (Lx-Lx2)) =	12.10	**ksi**	
fbult:	Bending Stress = Mbm x 6 / (t x (Lx-Lx2)^2) =	8.98	**ksi**	
	Combined normal stress = fault + fbult =	21.07	**ksi**	**OK <Phi* Fy**

Weld Stresses

Average ultimate stress =	8.86	**ksi/in.**	
Peak ultimate Stress =	10.11	**ksi/in.**	
Peak stress / Average stress =	1.14	**< 1.4**	If this ratio is less than 1.4,
Implied orientation of stresses =	0.89747	rad	size the weld for 1.4 times
Capacity increase for orientation =	1.35		the average ultimate stress.

Beam Web Stresses

Beam web stress =	19.67	**ksi**	**OK < Fybm**

Weld Design Gusset-to-Beam

Electrode =	***E70***	
Electrode correction (LRFD 2nd ed. p8-158) =	1	
Required fillet weld size =	6.62	**16ths in.**

Design of Gusset-to-Column Connection — **CONCLUSIONS**

Design a fillet weld on each side of the gusset plate to the column.

Interface Forces

	Column Orientation =	Strong	
Hcol:	Horiz. force at the gusset-column interface = Hc	96.50	**kips**
Vcol:	Vertical force at the gusset-column interface = Vc	153.32	**kips**
Mcol:	Moment at the gusset-column interface = Mc	-205.76	**in.-kips**

Gusset Stresses

Fvcap:	Gusset shear capacity = 0.9x0.6 x Fy =	19.44	**ksi**	
fvult:	Shear stress on gusset = Vcol / (t x (Ly-Ly2)) =	12.03	**ksi**	**OK < Fvcap**
fault:	Axial stress = Hcol / (t x (Ly-Ly2)) =	7.57	**ksi**	
fbult:	Bending Stress = Mcol x 6 / (t x (Ly-Ly2)^2) =	5.70	**ksi**	
	Combined normal stress = fault + fbult =	13.26	**ksi**	**OK <Phi* Fy**
	Implied orientation of stresses=	0.83436	**rad**	
	Capacity increase for orientation=	1.32		

COLUMN STRONG WAY

Design welds between the gusset and the column flange.

Strong-Way Weld Stresses

Average ultimate stress =	5.97	**ksi/in.**	
Peak ultimate Stress =	6.71	**ksi/in.**	
peak stress/average stress =	1.13	**< 1.4**	If this ratio is less than 1.4, size the weld for 1.4 times the average ultimate stress.

Column Web Stresses

Column web yielding stress =	11.37	**ksi**	**OK < Fy**

Weld Design

Electrode =	***E70***	
Electrode correction (LRFD 2nd ed. p8-158) =	1	
Required fillet weld size =	5.00	**16ths in.**

COLUMN WEAK WAY **Not Applicable**

Design welds between the gusset and the column web, and plates between column flanges.

Weak-Way Weld Stresses

fvweb:	Unit shear weld to web = Vcol/(2 x (Ly-Ly2)) =	N.A.	**kips/in.**	
Fvww:	Vert. shear cap. of col. web = 0.55 x Fy =	27.50	**ksi**	
	Column web stress =	0.00	**ksi**	**OK < Fvww**

Weld Design Gusset-to-Column

Electrode =	***E70***	
Electrode correction (LRFD 2nd ed. p8-158) =	1	
Required fillet weld size =	N.A.	**16ths in.**

Plate Design

Design plate to be welded to the gusset plate and the column flanges

PLfy:	Yield strength for the plate =	***36.00***	**ksi**	
PLt:	Plate thickness =	***0.50***	**in.**	
PLL:	Length of plate =	N.A.	**in.**	
PLf:	Force in the plate =	N.A.	**kips**	
Fvpl:	Ult. shear cap. of pl. = 0.55 x PLfy =	19.80	**ksi**	
PLfv:	Shear stress in plate = (PLf / (2 x PLL x PLt)) =	N.A.	**ksi**	**N.A.**
PLfb:	Bending stress in plate =	N.A.	**ksi**	**N.A.**

Weld Design Plate-to-Column/Gusset

Electrode =	***E70***	
Electrode correction (LRFD 2nd ed. p8-158) =	1	
Req'd fillet weld size of plate-to-gusset =	N.A.	**16ths in.**
Req'd fillet weld of plate-to-column flange =	N.A.	**16ths in.**

Summary

USSET PL 'A'						WELDS (16th's)		
Thk (in)	**Lcol (in)**	**Lbm (in)**	**Lbr (in)**	**Ld (in)**	**Lw (in)**	**A**	**B**	**C**
0.75	22	21.07	12	42	9	5.00	6.62	6.44

PLATES					
Ref.	**Thk (in)**	**Length (in)**	**idth (in)**	**Weld (16th)**	**Gap (in)**
F	0.5	N.A.	12.6	N.A.	

plate. Half of the force is transferred to the plate by each half of the tube section. The centroid of the half section is no longer at the centroid of the plate, but rather is offset from the face of the plate toward the remaining wall of the tube section. The eccentricity between this centroid and the weld to the plate creates bending along the length of the welds. Of course, equal and opposite bending exists on the other side of the plate resulting in no net bending on the plate provided the tube is slotted along its centerline. The welds must be sufficiently strong to resist this bending stress. Damage to similar connections was found after the Northridge earthquake where the lack of sufficient weld to resist this bending resulted in the sides of the tube peeling away from the gusset plate. Welds may be strengthened by either increasing their thickness or length. Although increasing the length is the most efficient locally, it may increase the gusset-plate size and connections to the beam and column depending on the configuration.

The connection of the brace to the gusset plate is also subject to block shear. For the tube steel brace, the plate may yield around the perimeter of the full tube section (two lines of shear and one of tension) or along each weld line in shear (four lines of shear). The tube section may also yield along the tube walls (four lines of shear). Other section types would have similar mechanisms.

Another consideration is the reduced net section of the brace resulting from the slot that is provided for the weld between the gusset plate. This net section has caused failure of tested braces, and should therefore be reinforced. Common means of such reinforcement are added plates that are shop welded to the vertical faces of the HSS members.

Gusset-plate design. The gusset plate may either allow for out-of-plane rotation of buckling braces or may restrain the brace elastically. The design philosophy chosen will affect the slenderness ratio used for the brace. If the connection is not capable of restraining the rotation of the buckling brace, an effective length factor, K, of 1.0 is used. If, however, the connection can restrain the bending demands of the buckling brace a smaller value of K may be used. The connection must then be strong enough to restrain the bending capacity of the brace taken as $1.1R_yM_p$ about each axis. Although more efficient brace members may be utilized, more robust connections will be required which will at least partially offset the material savings.

An accepted design methodology for the gusset plate which allows member end rotation was researched by Goel and has been adopted by the AISC *Seismic Provisions*. The provisions require that the brace maintain a minimum distance of 2 times the thickness of the plate from the anticipated line about which the plate will yield flexurally as the brace buckles. This line is assumed to occur between points of restraint such as the end of the gusset-to-beam connection and gusset-to-column connection. Stiffener plates may also be used to support the plate. The design should also maintain this line perpendicular to the axis of the brace to ensure the brace will buckle perpendicular to the plane of the frame.

This example allows the buckling to occur in the out-of-plane direction while it is assumed that in-plane buckling is restrained and will not control the design. If rectangular sections with largely differing properties were chosen, the capacity in each direction would need to be investigated to determine which controls the design. An effective width of plate can be calculated using Whitmore's method presented in AISC and is checked for tension and compression. The tension capacity of the gusset is conservatively estimated at $A_s F_y$ while the brace ultimate capacity is utilized. The gusset plate is checked for compression strength in an area where it is restrained by the beam, column, and/or stiffener plates on all sides but one: along the buckling line. The true effective buckling length is complicated at best, but conservatively may be estimated at 0.8. Alternately, 1.0 between hinge locations may also be used.

Gusset-plate–to–beam-and-column connection. The forces imparted from the gusset plate to the face of the beam and column are obtained from the analysis using the uniform force method. Unless specifically optimized otherwise, each connection will see axial, shear, and bending forces. The plate, as well as the welds, are designed to remain elastic under these forces. The capacity of the weld may be checked in a number of ways. It is conservative to calculate an effective eccentricity of the shear force to the weld and add it vectorially to the axial force resulting in an effective force with an eccentricity and angle to the weld. The AISC charts for eccentrically loaded weld groups may then be used to determine the weld capacity.

Beam-to-column connection. Connection of the beam to the column is designed to transfer the resulting axial, shear, and bending demands on the beam. Due to the connections' highly restrained configuration from the gusset plate(s), this connection must be very stiff to adequately resist the forces. Moment-frame type connections consisting of groove-welded flanges and either welded or bolted webs using slip-critical bolts are typically used. The web and flange connections are sized to develop their share of the forces at the joint. It is typically sufficient to use full-penetration-welded flanges and webs.

The last page of Example 5.2 summarizes the design and Fig. 5.19 shows the final detail of the connection.

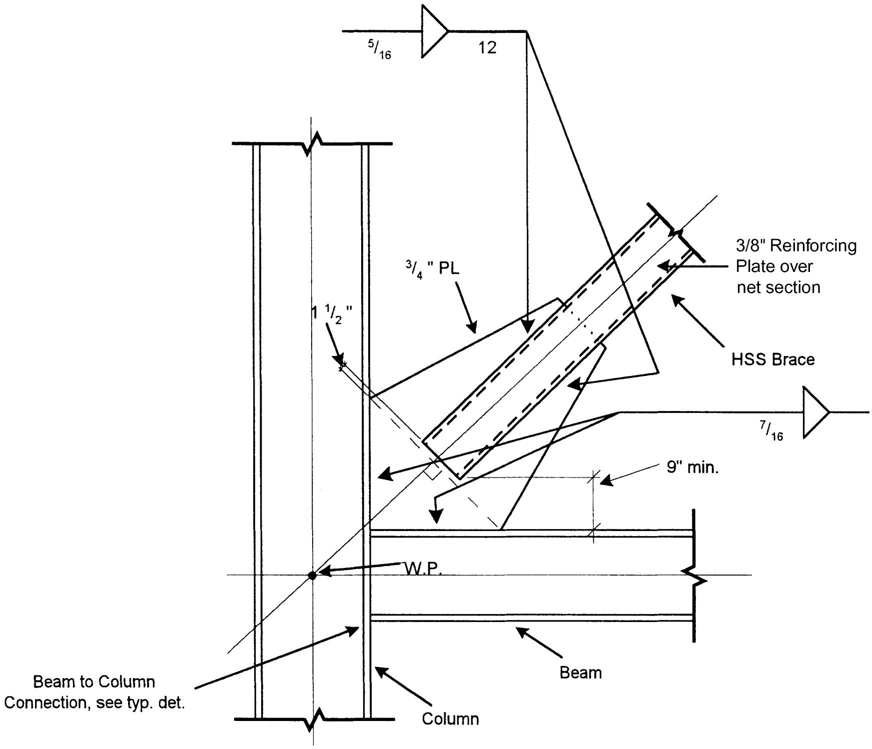


Figure 5.19 Brace connection.

5.5 Eccentrically Braced Frames

Eccentrically braced frames (EBFs) are braced frame systems which utilize a link beam created by the eccentric connection of the diagonal brace or braces. The system provides energy dissipation through inelastic deformation of the link. The link either yields in shear (short links) or in bending (long links), while the beams, columns, and braces in the system remain elastic.

The design of the connections in an EBF is very similar to that of the SCBF. The methodology used in Example 5.2 could be used to design a brace-to-beam or brace-to-column connection in an EBF with the following exceptions. First, where the SCBF was designed based on the capacity of the brace, in an EBF the expected capacity of the link is used to size the brace and beam connections. Second, since the brace is not intended to yield, providing the $2t$ buckling line is not necessary. Finally, the eccentricity of the brace to the beam creates large bending demands in the link which are resisted by the beam outside of the link and by the brace member. Although braces have traditionally been considered pinned, in an EBF a brace can attract significant bending due to their fixed connections which must be accounted for in the design of the brace and its connection to the beam and/or column. The additional bending on the gusset plate may be superimposed with the force distribution obtained from the uniform force method.

5.6 Buckling Restrained Braced Frames

Like SCBF, connections between members of BRBFs are intended to be able to force inelastic behavior to occur in the braces. AISC 341 requires that the connections have a required strength of 1.1 times the expected strength of the brace. For BRBFs the expected strength of the brace is likely to be controlled by the compression yielding, which is generally on the order of 10% higher than the tension capacity. Because BRBFs are not subject to brace buckling, gusset plate designs that rely on folding on the yield line (see Example 5.2) are not required. Force distribution calculations using the uniform force method would still apply, however.

5.7 Special Plate Shear Walls

For SPSW systems, the concept for connection design is identical to the other ductile steel systems. The connections between the plates and the boundary elements are designed to develop the expected capacity of the plate, recognizing the characteristic angle of plate yielding. In addition, the connections between the HBE and VBE are required to be fully restrained moment resisting connections designed to meet the requirements for OMFs at the expected yield capacity of the HBE members. Further, the shear capacity of this connection must be able to transfer the vertical shear induced by the yielded wall plates in addition to the shear that is generated by fully yielding of the HBE as a moment frame member. The shear induced by the yielded wall plates can become especially significant at the top and bottom stories of the walls, and where there is a transition in thickness of the wall plate. At other levels, this shear is basically cancelled out by the wall plates above and below the HBE.

5.8 Other Connections in Seismic Frames

In addition to the connections between primary members of the seismic load-resisting system that have been discussed in this chapter, there are a number of other connections that are critical to the seismic performance of steel systems. The first is the splice of seismic frame columns. The AISC *Seismic Provisions* have a series of requirements to help ensure that these splices are able to resist all the forces necessary to develop the intended inelastic performance of the system without fracture. The first of these requirements is that the splices be designed to resist the amplified seismic loads for any tensile stresses. In addition partial joint penetration welds are more susceptible to fracture, they are required to be designed for twice the amplified seismic load. And, in all cases, the connection must be able to develop at least 50% of the expected tensile capacity of the smaller column. In addition to these requirements that apply to all steel systems, there are additional requirements for the highly ductile systems. For example, the requirements for SMF systems effectively require column splices that develop the expected tensile capacity of the smaller column, with a shear capacity sufficient to form a plastic hinge in the column. For the braced frame systems (other than OCBF), the column splices are generally required to resist a flexural strength of at least 50% of the smaller column, with a shear capacity to form a plastic hinge in the column.

The connection of the column to the base plate is a special case of the typical column splice. It is critical that the design engineer provide a base connection that is not subject to fracture, since it is well understood that these connections are often subject to inelastic behavior in order a full plastic mechanism to be developed in a well-proportioned steel frame structure. However, the design requirements in previous editions of building codes have not adequately addressed these connections, and the transition between steel and the concrete foundation elements, and the transition between concrete foundation and the supporting soils. AISC 341 now has a section that defines the requirements for the design of the column base and the anchorage of the base into the concrete foundation. Essentially, the requirements cause the base connection to be designed for the same force that the column has been designed for, in flexure, shear, and axial force.

In addition to the requirements for splices for columns that are part of the lateral resisting system, the AISC *Seismic Provisions* require a check of the columns that are not deemed to be part of the SLRS. This check verifies a minimum shear capacity needed to generate a plastic hinge in the column over a single story height. This requirement is the result of evaluating the beneficial effect that these nonseismic frame columns can have in the overall inelastic performance of steel framing systems. The additional capacity of these columns can help to avoid the formation of story mechanisms that can be very detrimental to seismic performance.

The other connections of note for good seismic performance are those provided for out-ofplane stability. Providing out-of-plane stability is critical to ensuring the expected performance of seismic systems that are anticipated to undergo substantial story drifts and large inelastic demands in the event of a design level earthquake. The AISC *Seismic*

Provisions include a series of requirements for the various systems. These requirements include both strength and stiffness checks for the bracing elements and connections, based on the provisions of the main AISC Specification (AISC 360). The design force for these connections varies depending on whether or not the stability bracing is located adjacent to a plastic hinge. Significantly higher bracing forces develop at these hinge locations, on the order of 6% of the beam flange capacity. Proper seismic performance of the entire frame necessitates that this stability bracing be provided.

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Chapter

6

Structural Steel Details

David R. Williams, P.E.

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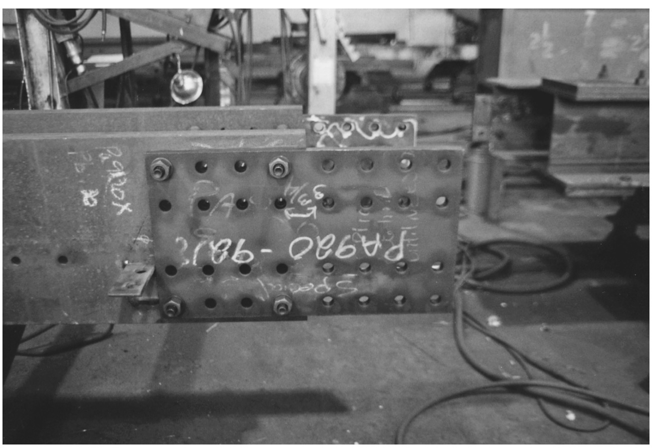
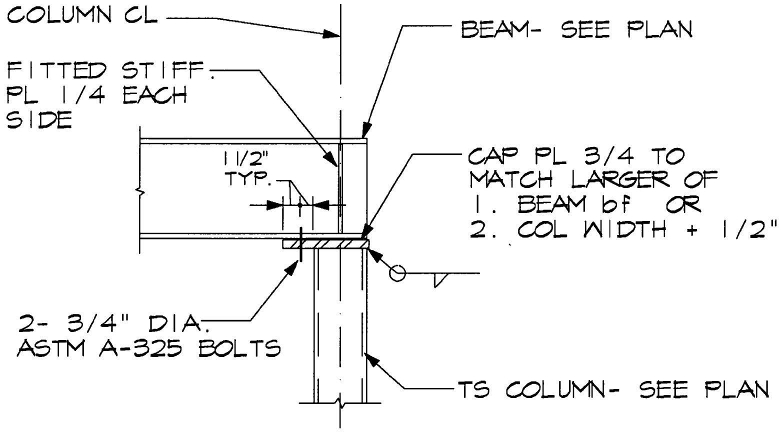
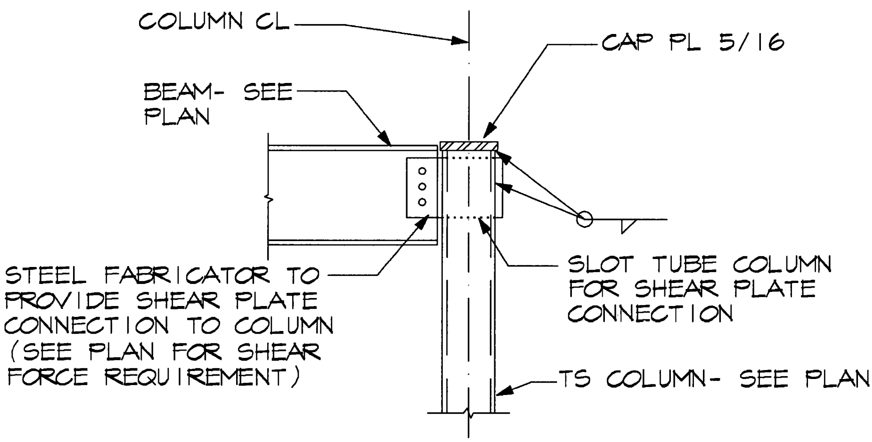


(*Courtesy of The Steel Institute of New York.*)



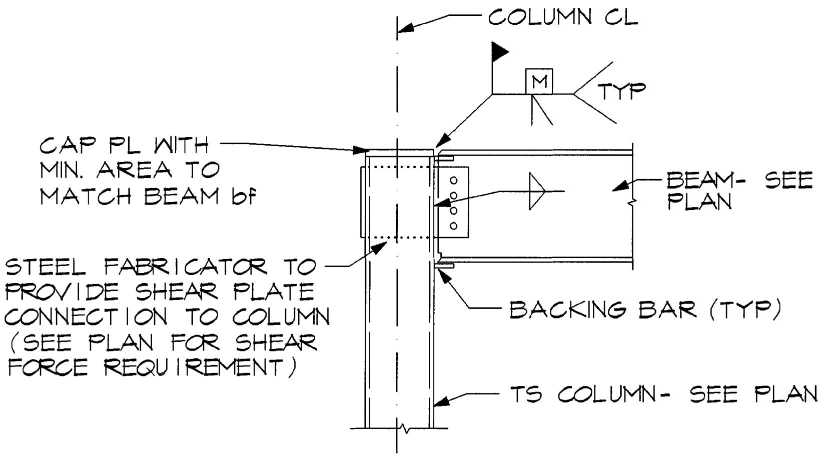
BEAM FRAMES OVER TS COLUMN



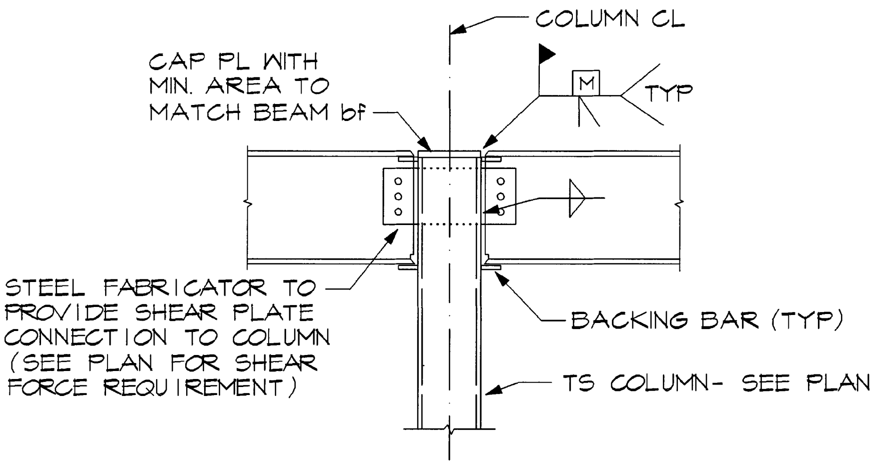
BEAM FRAMES INTO TS COLUMN

BEAM SHEAR CONNECTION DETAILS

NOT TO SCALE (DETAIL T5–CSH1)



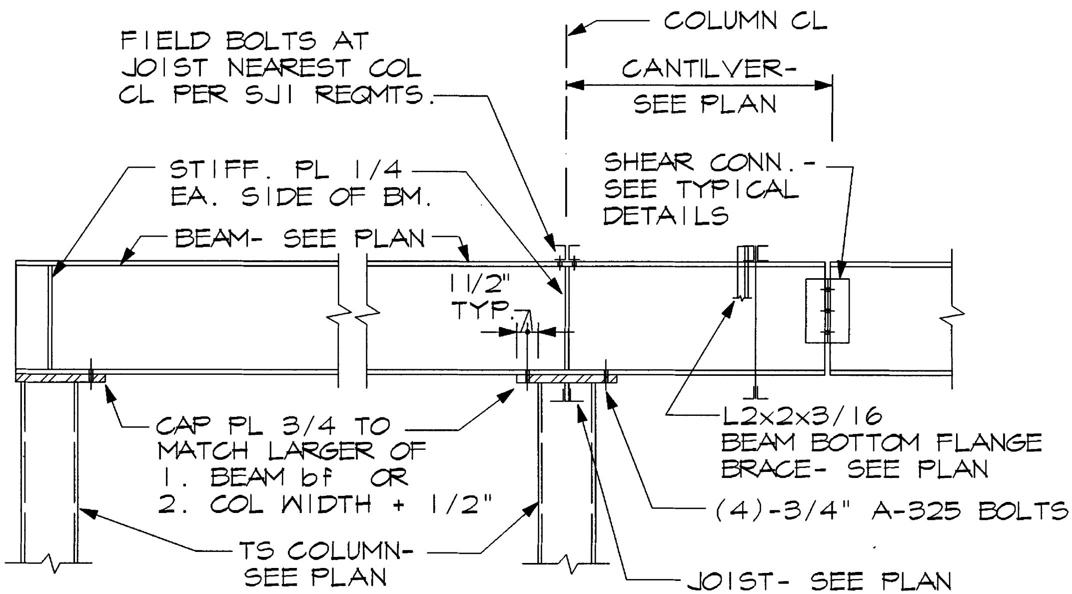
BEAM FRAMES ONE SIDE OF TS COLUMN



BEAM FRAMES BOTH SIDES OF TS COLUMN

BEAM MOMENT CONNECTION DETAILS

NOT TO SCALE (DETAIL T5-CM01)



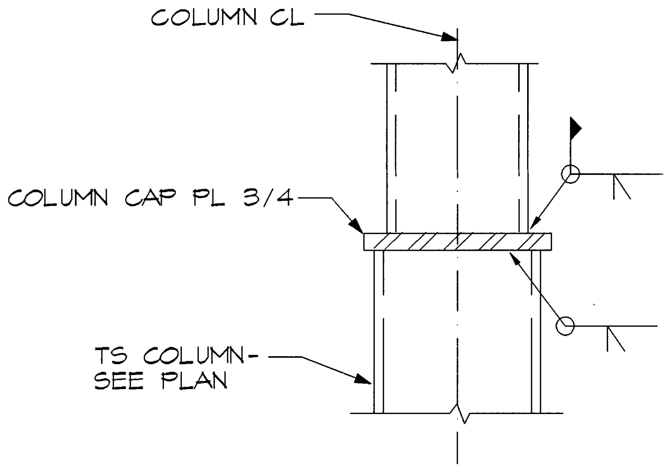
END TS COLUMN

INTERIOR TS COLUMN

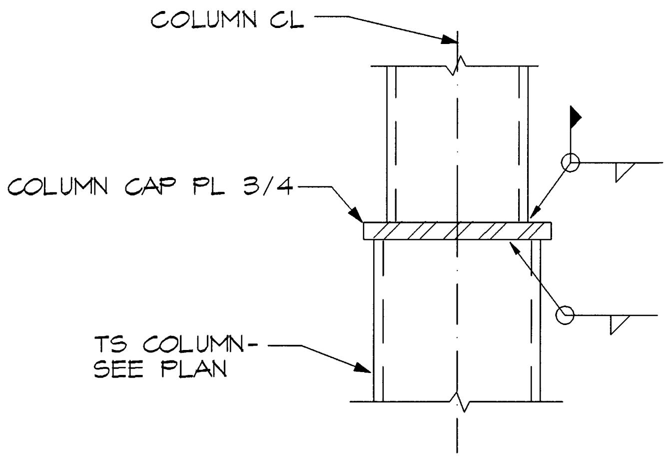
TYPICAL CANTILEVER ROOF BEAM DETAILS

NOT TO SCALE

(DETAIL T5–CBM1)



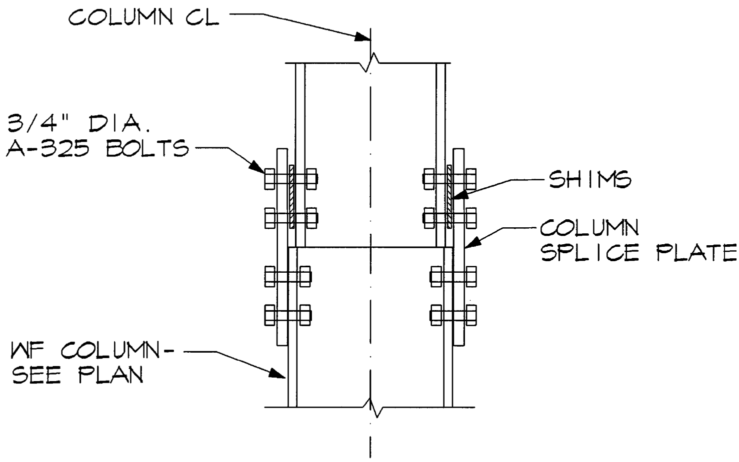
TS COLUMN SPLICE, FILLET WELDED



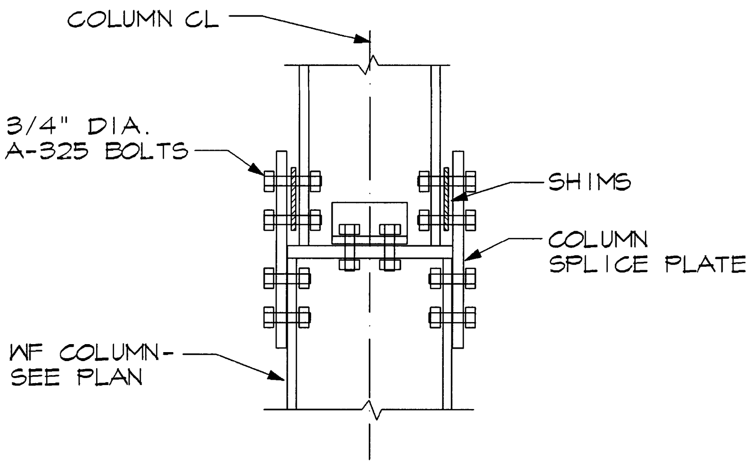
TS COLUMN SPLICE, PENETRATION WELDED

TYP. TS COLUMN SPLICE DETAILS

NOT TO SCALE (DETAIL T5–COL1)



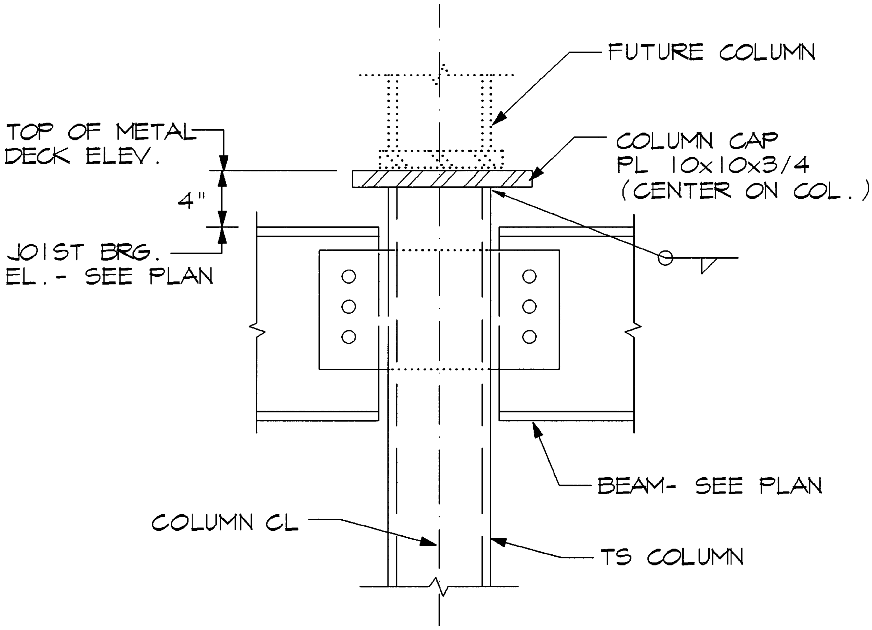
WF COLUMN SPLICE, BOLTED



WF COLUMN SPLICE, BUTT PLATE

TYP. WF COLUMN SPLICE DETAILS

NOT TO SCALE (DETAIL T5–COL2)

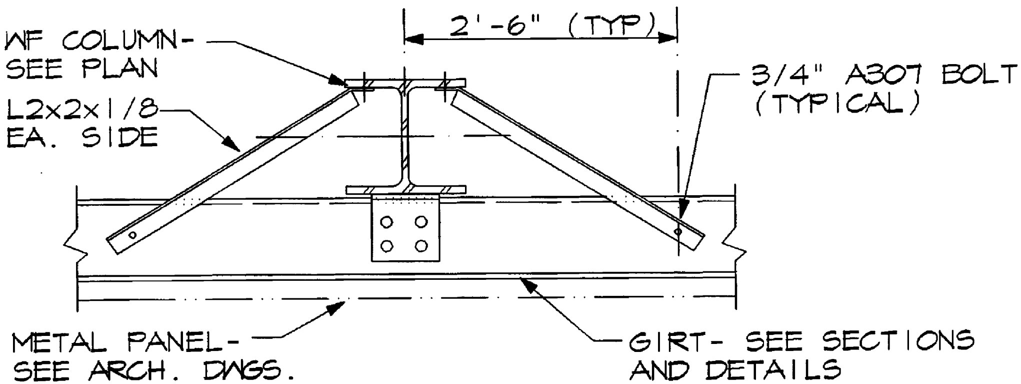


TYP. FUTURE COLUMN CONNECTION

NOT TO SCALE (DETAIL T5–COL3)

NOTES:

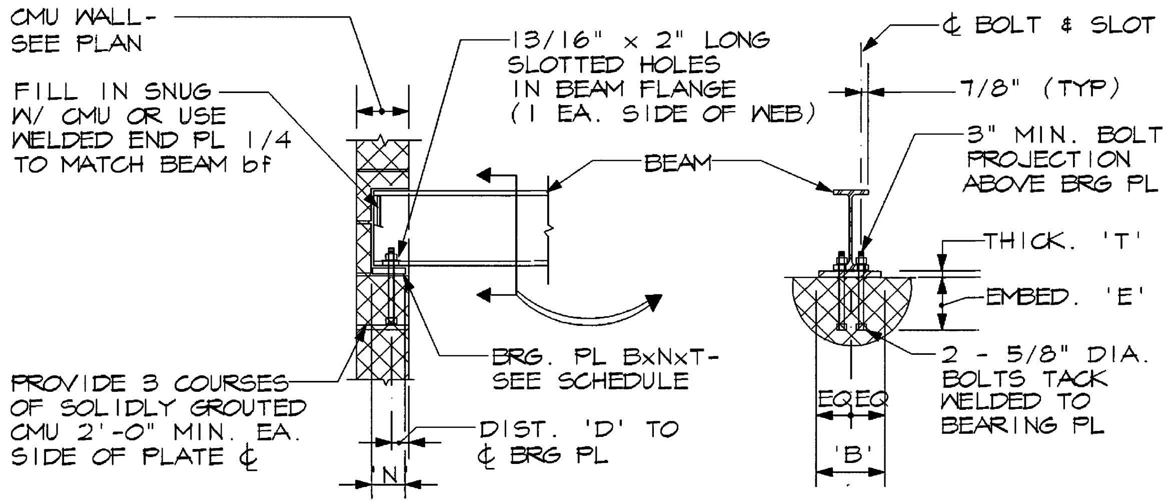
1. SEE WALL ELEVATIONS DETAILS FOR LOCATIONS.
2. DETAIL AT PERIMETER COLUMNS IS SHOWN. DETAIL AT CORNER COLUMNS IS SIMILAR.



TYPICAL COLUMN BRACE DETAIL

NOT TO SCALE (DETAIL T5-BRCE1)

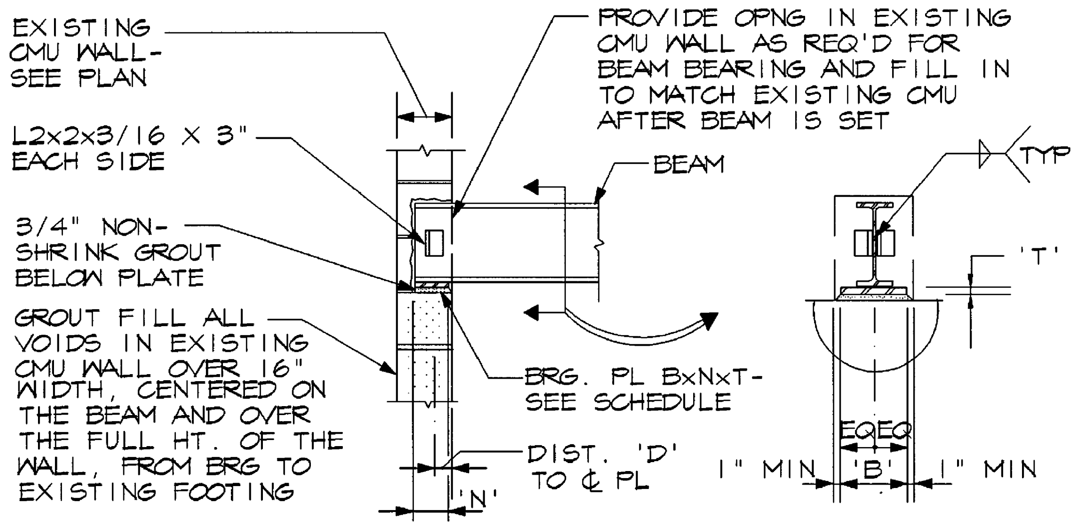
NOTE: DETAIL IS SHOWN FOR BEAMS PERPENDICULAR TO WALL, PROVIDE SIMILAR DETAIL FOR BEAMS PARALLEL TO WALL.



"FIXED" CONNECTION

TYPICAL BEAM BEARING ON MASONRY DETAILS

NOT TO SCALE (DETAIL T5–CMU1)

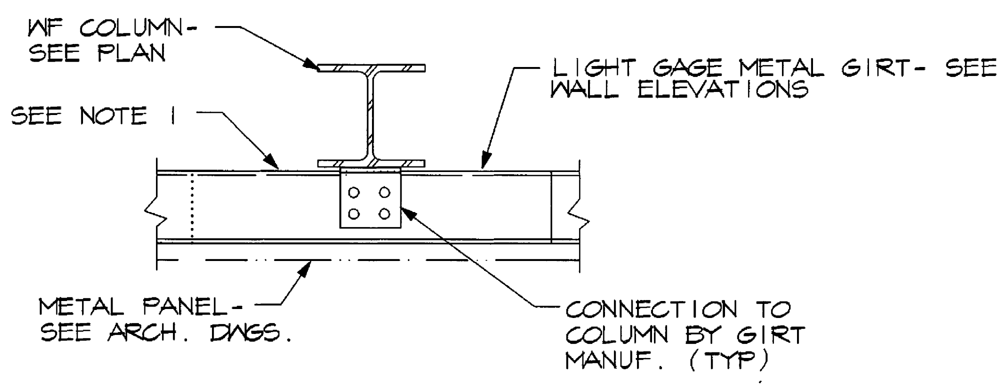


TYPICAL BEAM BEARING ON EXISTING MASONRY DETAILS

NOT TO SCALE

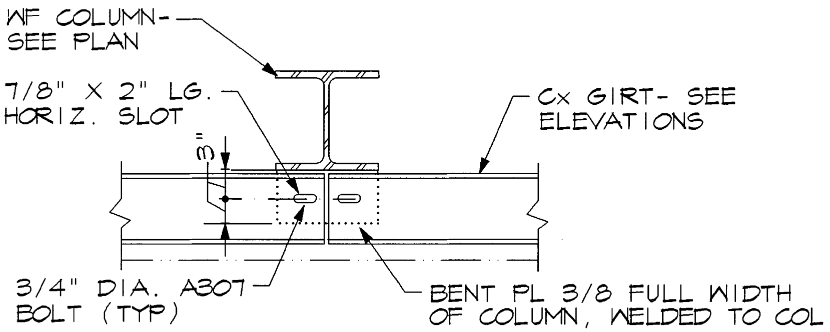
(DETAIL T5–CMU3)

NOTES:
1. LAP GIRTS AT COLUMN AS REQUIRED BY THE MANUFACTURER.
2. PROVIDE GIRT WEB STIFFENERS AS REQUIRED BY THE MANUF.

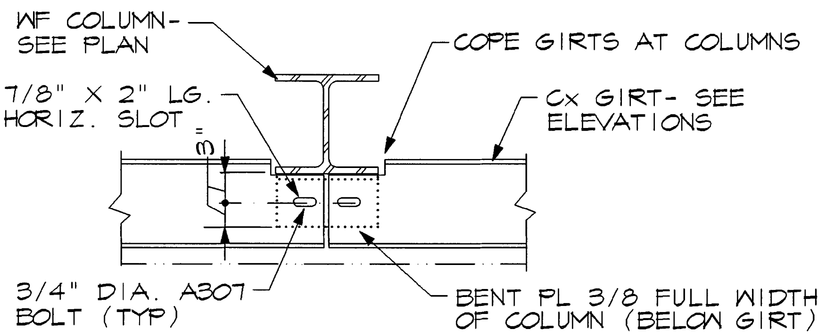


TYPICAL LIGHT-GAGE METAL GIRT CONNECTION DETAILS

NOT TO SCALE (DETAIL T5-GIRT1)



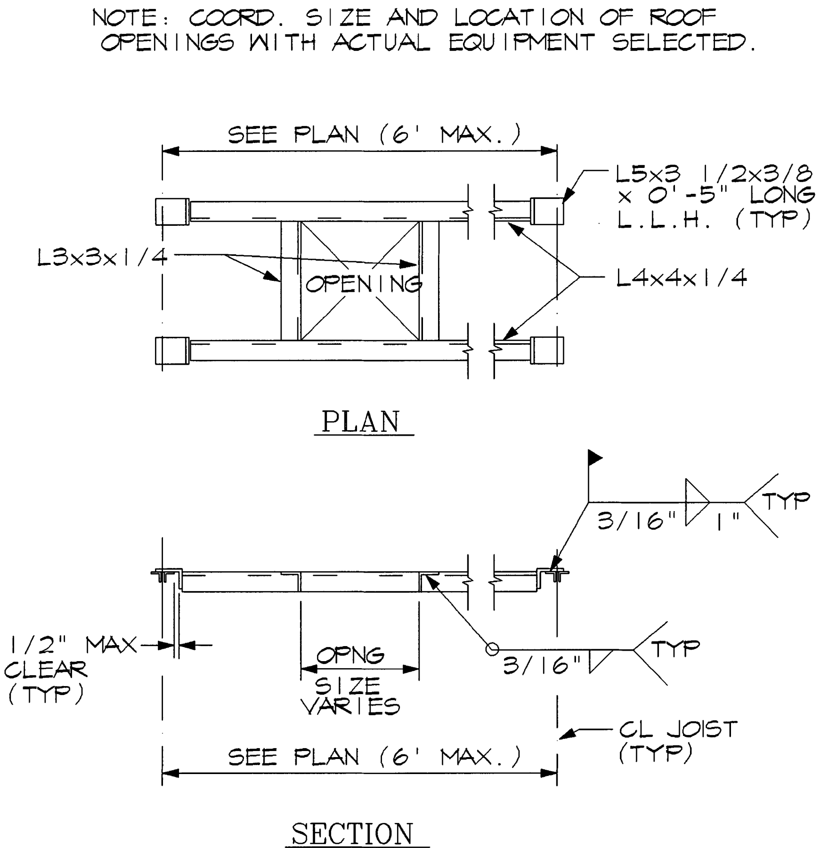
AT TYP. GIRTS



AT COPED GIRTS

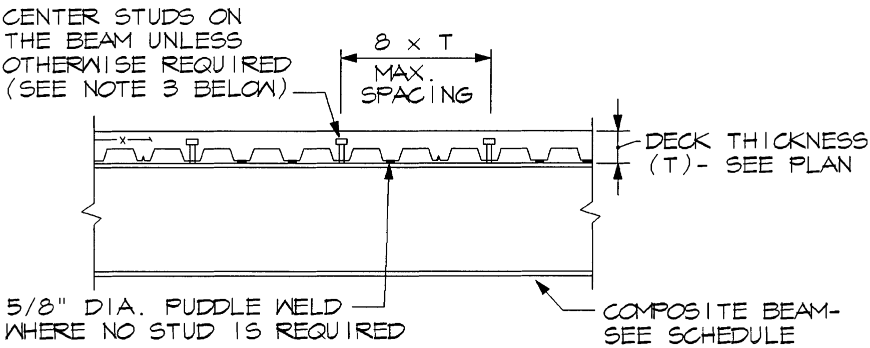
TYP. GIRT CONNECTION DETAILS

NOT TO SCALE (DETAIL T5–GIRT4)



TYP. ROOF OPENING DETAIL

NOT TO SCALE (DETAIL T5–R01)



DECK IS PERPENDICULAR OR SKEWED TO BEAM

NOTES:

1. THE MIN. NUMBER OF STUDS FOR EACH BEAM IS SHOWN IN THE COMPOSITE BEAM SCHEDULE.

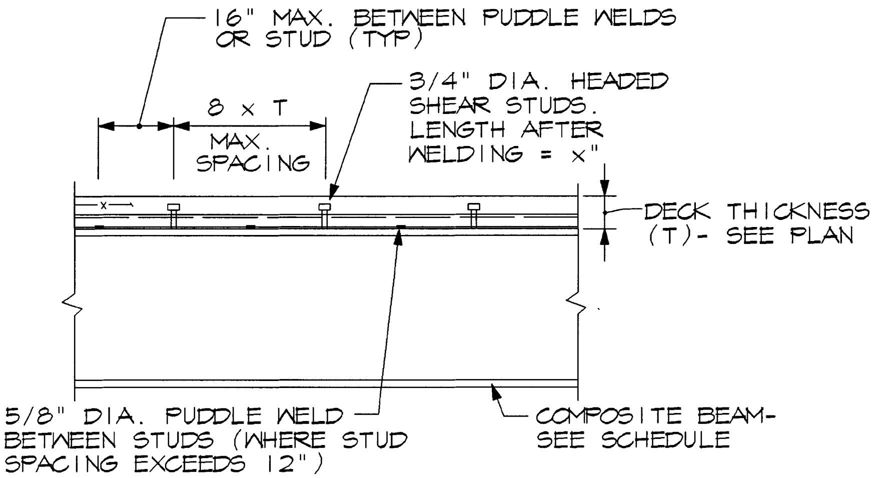
2. SPACE STUDS AS EVENLY AS POSSIBLE IN AVAILABLE DECK FLUTES. WHERE STUD SPACING EXCEEDS THE MAX. SPACING ALLOWED, PROVIDE ADDITIONAL STUDS TO SATSIFY THE SPACING REQUIREMENTS.

3. WHERE THE NUMBER OF STUDS EXCEEDS THE NUMBER OF FLUTES, PROVIDE TWO STUDS IN EVERY OTHER FLUTE, STARTING AT EACH END OF THE BEAM. THE TRANSVERSE SPACING BETWEEN TWO STUDS IN A SINGLE FLUTE SHALL BE 4 x STUD DIAMETERS (MIN.).

4. SEE THE COMPOSITE BEAM SCHEDULE FOR ADDITIONAL REQUIREMENTS. TURN THE NATURAL BEAM CAMBER UP.

TYP. COMPOSITE BEAM ELEVATION

NOT TO SCALE (DETAIL T5–COMP1)



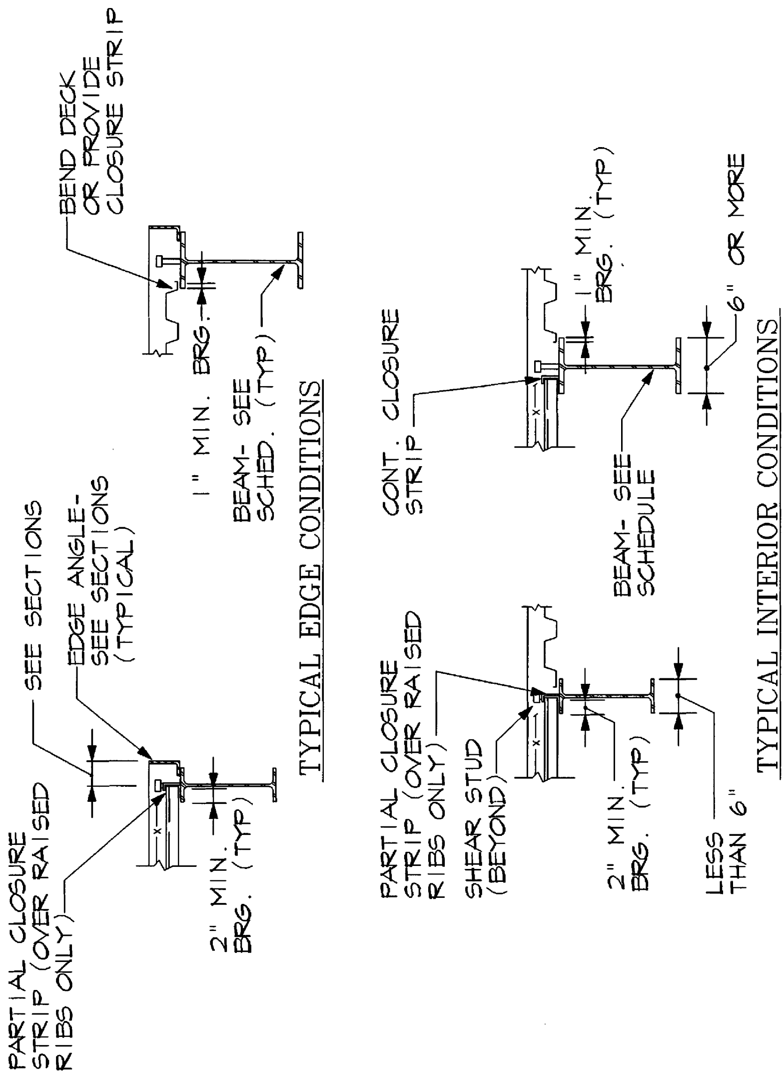
DECK IS PARALLEL TO BEAM

COMPOSITE BEAM SCHEDULE					
MARK	BEAM	MAX. END REACTION	MIDSPAN CAMBER	NUMBER OF STUDS	REMARKS
CB-1	W10x15	11 KIPS	—	12	INTERIOR
CB-2	W10x15	8 KIPS	—	8	INTERIOR
CB-3	W10x26	13 KIPS	—	12	INTERIOR
CB-4	W10x15	8 KIPS	—	3	CANTILEVER
CB-5	W10x26	13 KIPS	—	5	CANTILEVER
CB-6	W10x12	7 KIPS	—	3	CANTILEVER
CB-7	W18x76	86 k(@RT)	0.6"	22	GIRDER
CB-8	W16x40	36 k(@RT)	—	16	GIRDER

NOTE: NATURAL CAMBER OF BEAM SHALL BE TURNED UP.

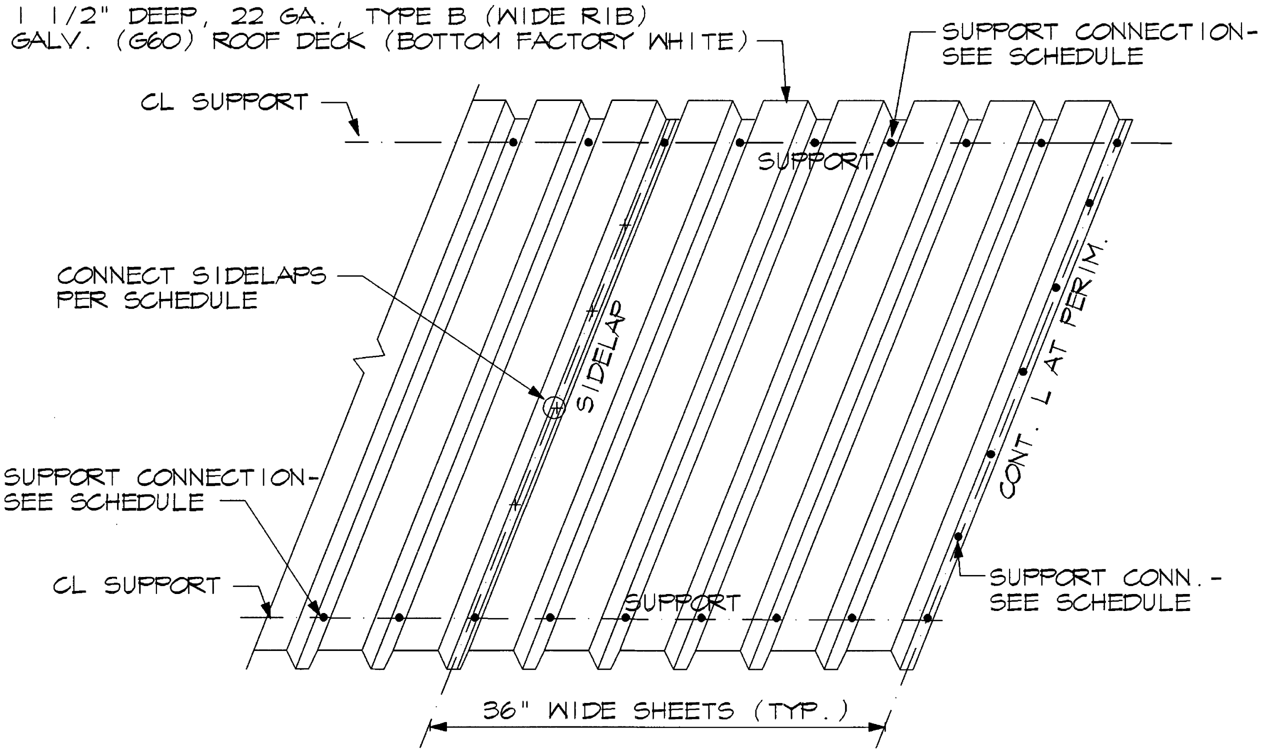
TYP. COMPOSITE BEAM ELEVATION

NOT TO SCALE (DETAIL T5–COMP2)



TYPICAL COMPOSITE DECK CLOSURE DETAILS

NOT TO SCALE (DETAIL T5-COMP3)

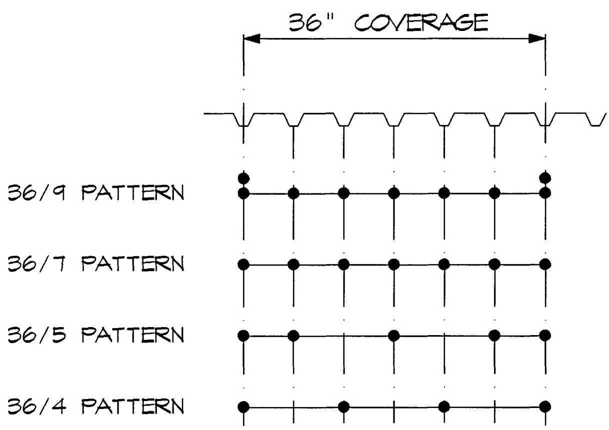


TYPICAL ROOF DECK ATTACHMENT DETAIL

NOT TO SCALE

(DETAIL T5–RDIA2)

ROOF DECK CONNECTION SCHEDULE	
ZONE	DECK CONNECTION
ZONE 1 (DIAPH. CAPACITY= 470#/LF)	WELDS IN 36/7 PATTERN (6)-#10 TEKS @ SIDELAPS
ZONE 2 (DIAPH. CAPACITY= 290#/LF)	WELDS IN 36/7 PATTERN (3)-#10 TEKS @ SIDELAPS
ZONE 3 (DIAPH. CAPACITY= 290#/LF)	WELDS IN 36/4 PATTERN (3)-#10 TEKS @ SIDELAPS

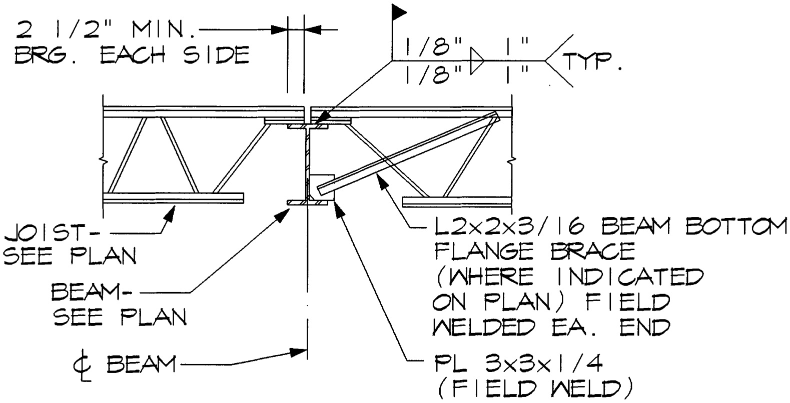


ROOF DECK ATTACHMENT PATTERNS

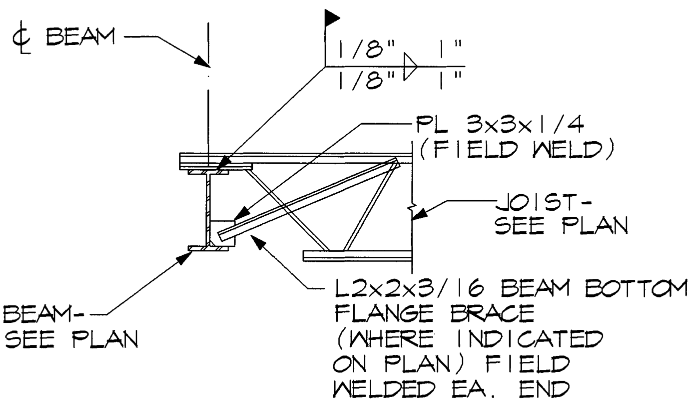
NOT TO SCALE (DETAIL T5–RDIA4)

NOTES:

1. AT JOIST NEAREST (OR ON) COLUMN CL CONNECT JOISTS WITH FIELD BOLTS PER S.J.I. REQUIREMENTS.
2. OFFSET JOISTS IF BEAM FLANGE IS LESS THAN 5" AND PROVIDE MINIMUM OF 2 1/2" JOIST BEARING.



AT INTERIOR BEAMS



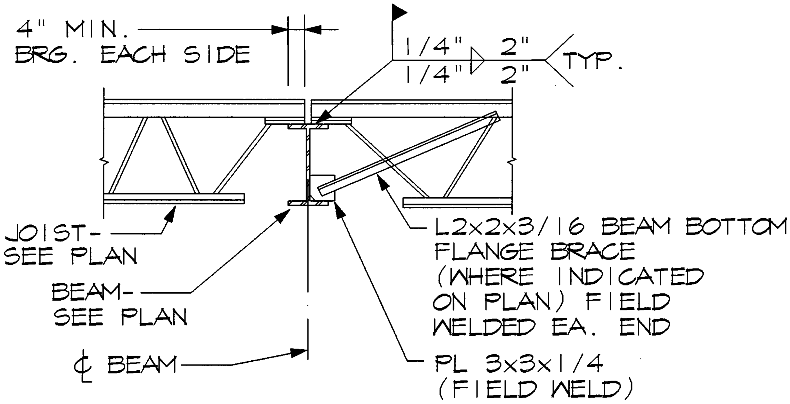
AT PERIMETER BEAMS

TYP. K-JOIST CONNECTION DETAILS

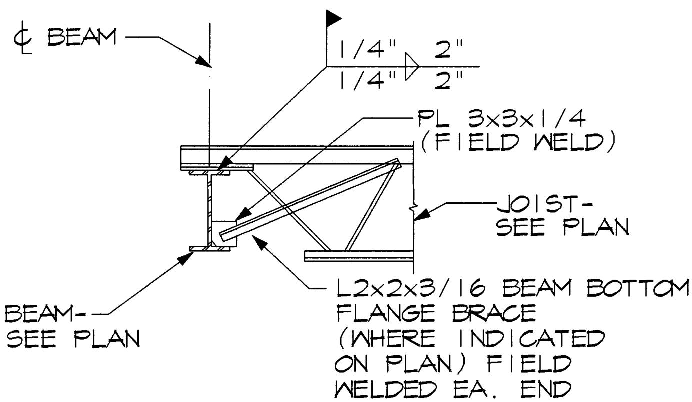
NOT TO SCALE (DETAIL T5-J1)

NOTES:

1. AT JOIST NEAREST (OR ON) COLUMN CL, CONNECT JOISTS WITH FIELD BOLTS PER S.J.I. REQUIREMENTS.
2. OFFSET JOISTS IF BEAM FLANGE IS LESS THAN 8" AND PROVIDE MINIMUM OF 4" JOIST BEARING.



AT INTERIOR BEAMS

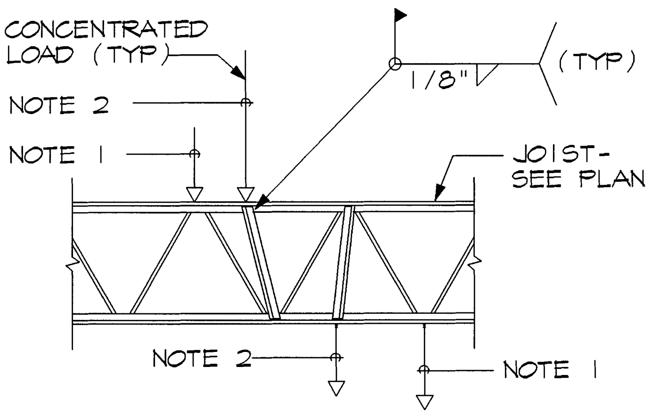


AT PERIMETER BEAMS

TYP. LH–JOIST CONNECTION DETAILS

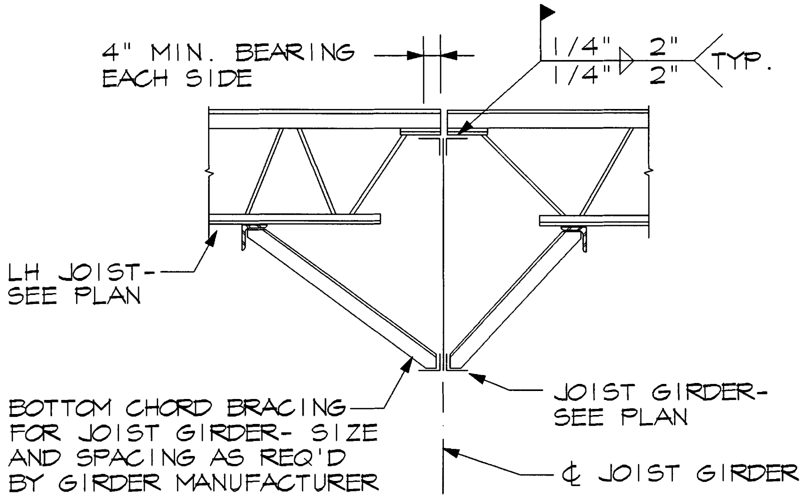
NOT TO SCALE (DETAIL T5–J2)

NOTES:
1. CONCENTRATED LOAD LOCATED AT JOIST PANEL POINT LOCATION - NO ADDITIONAL ANGLES REQUIRED.
2. CONCENTRATED LOAD (100 LBS. OR HEAVIER) NOT LOCATED AT JOIST PANEL POINT LOCATION - PROVIDE ⅃L1x1x1/8 TO PANEL POINT AS SHOWN.

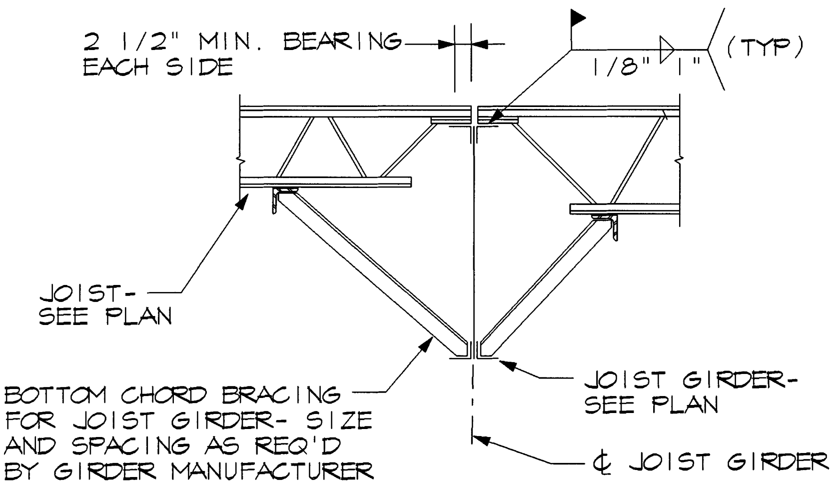


TYP. CONCENTRATED LOAD DETAIL

NOT TO SCALE (DETAIL T5–J5)



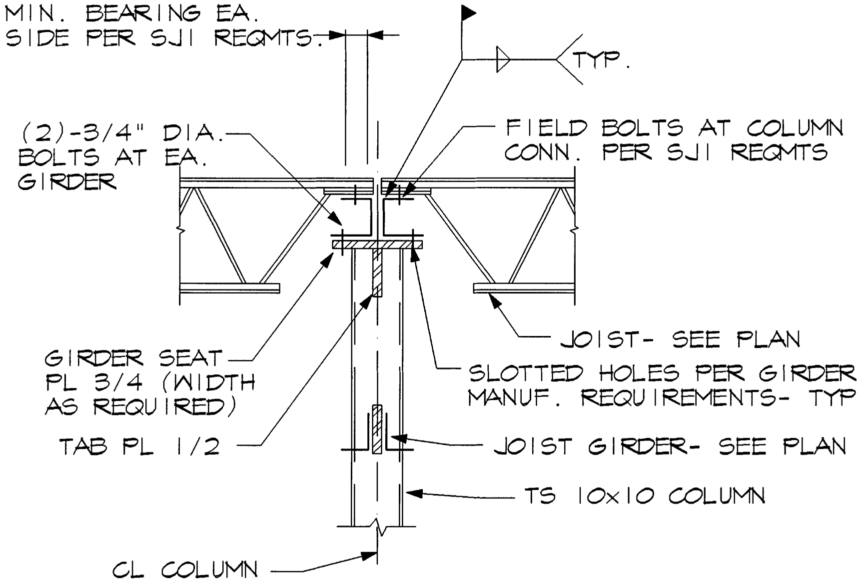
LH–SERIES JOISTS



K– SERIES JOISTS

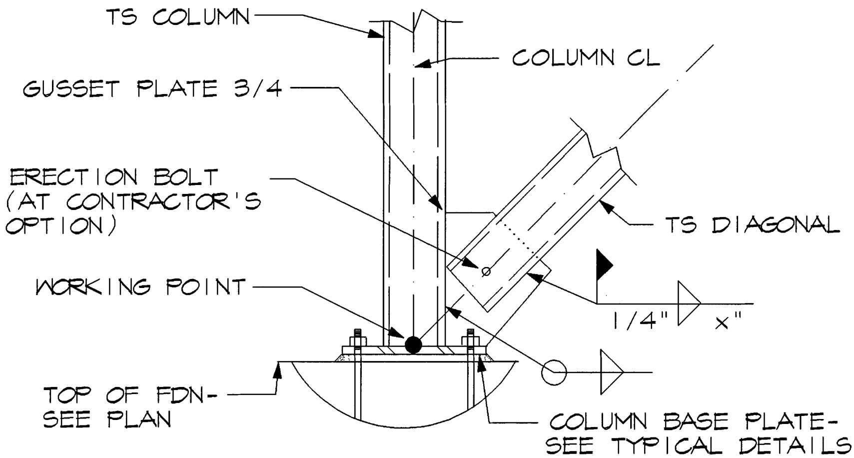
TYP. JOIST TO GIRDER CONNECTION

NOT TO SCALE (DETAIL T5–JG1)

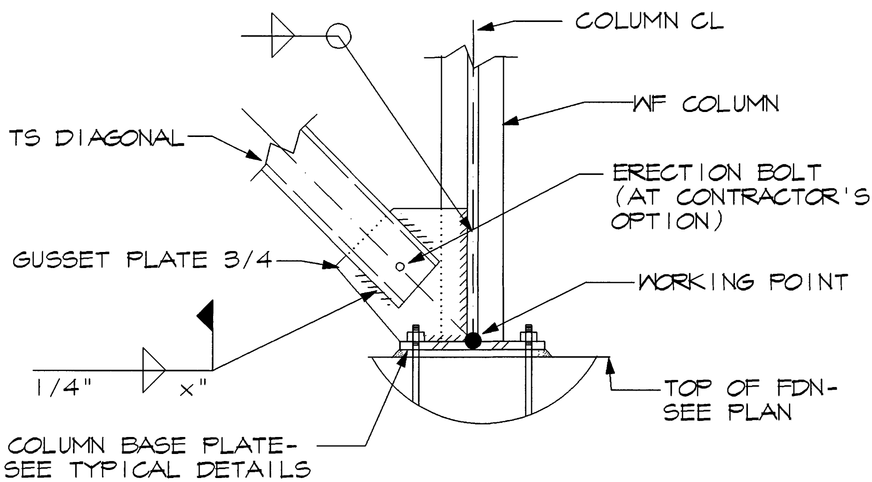


TYP. DETAIL AT INTERIOR COLUMN CL

NOT TO SCALE (DETAIL T5–JG2)



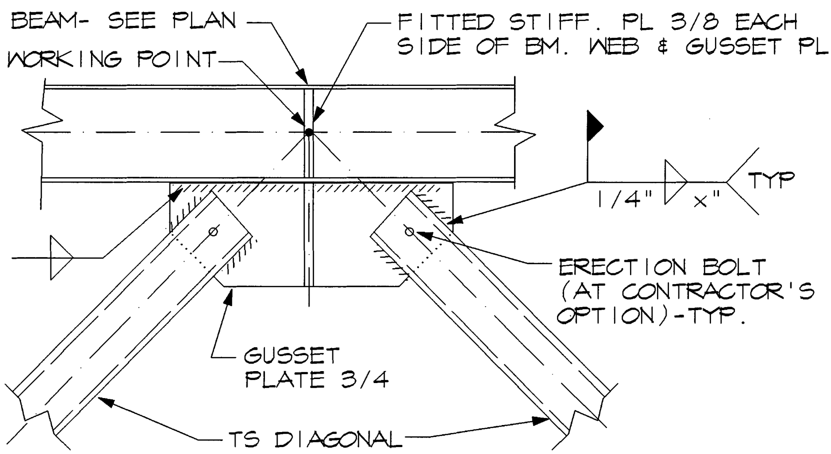
DETAIL A, AT TS COLUMN



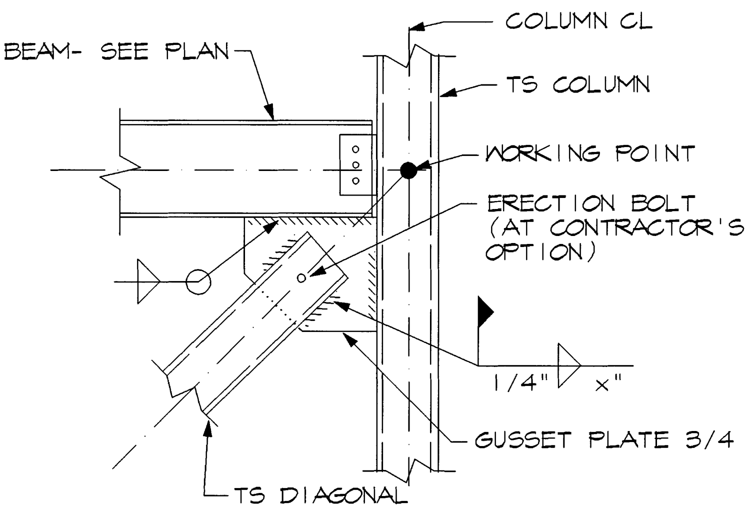
DETAIL A, AT WF COLUMN

BRACED BAY DETAILS

NOT TO SCALE (DETAIL T5–BAYD1)



DETAIL B

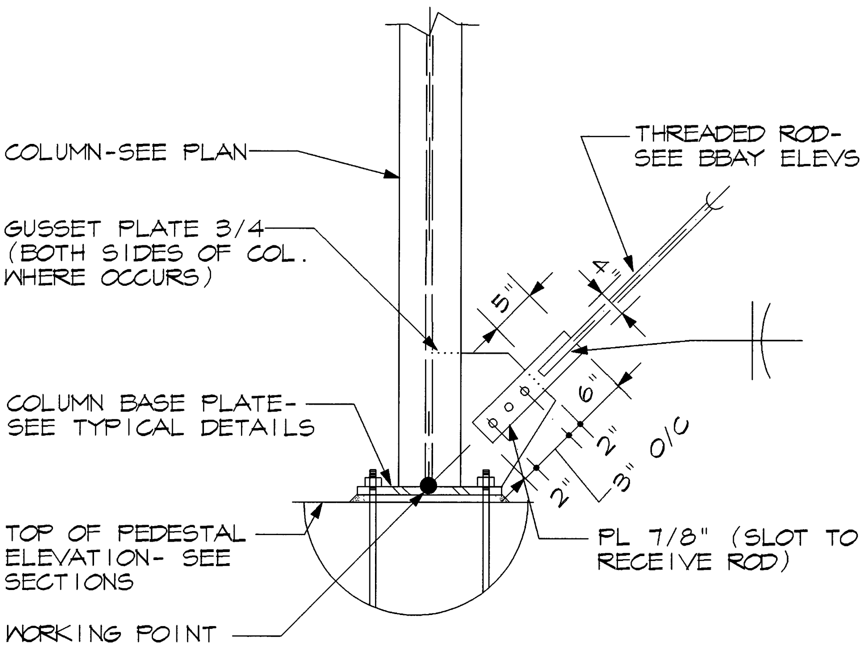


DETAIL C

BRACED BAY DETAILS

NOT TO SCALE (DETAIL T5–BAYD2)

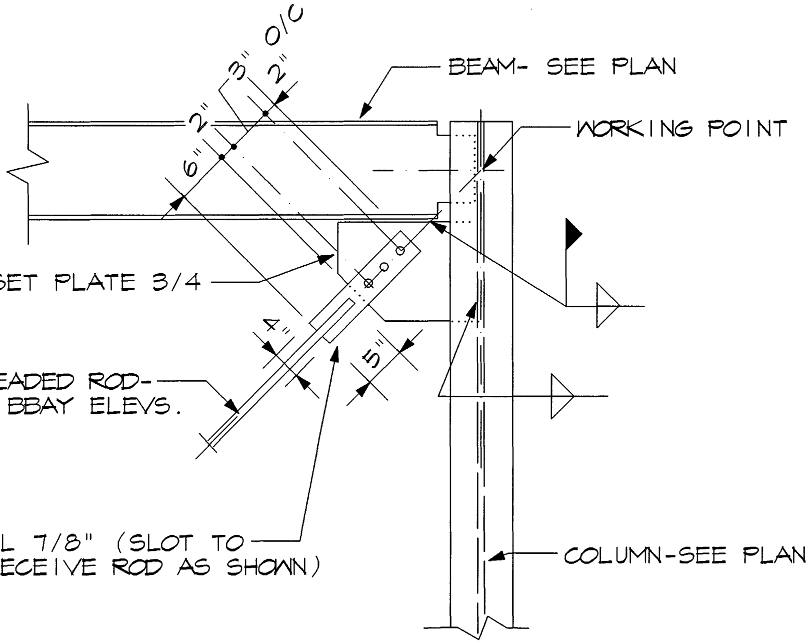
NOTE: PROVIDE BOLTED CONNECTION BETWEEN PL 7/8" AND THE 3/4" GUSSET PER THE FORCES INDICATED ON THE BRACED BAY ELEVATIONS



DETAIL A, USING THREADED RODS

BRACED BAY DETAILS

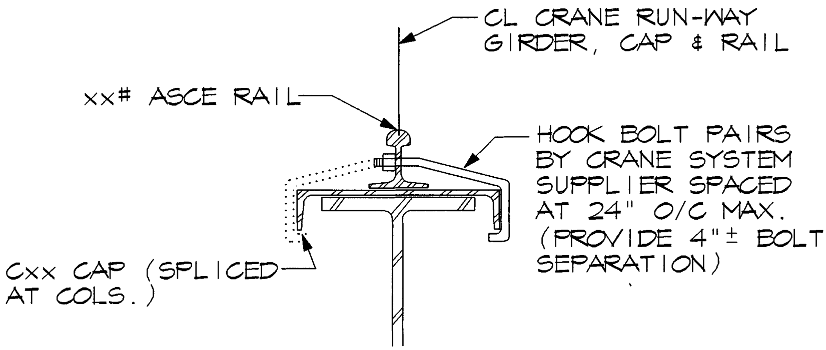
NOT TO SCALE (DETAIL T5–BAYD4)



DETAIL C, USING THREADED RODS

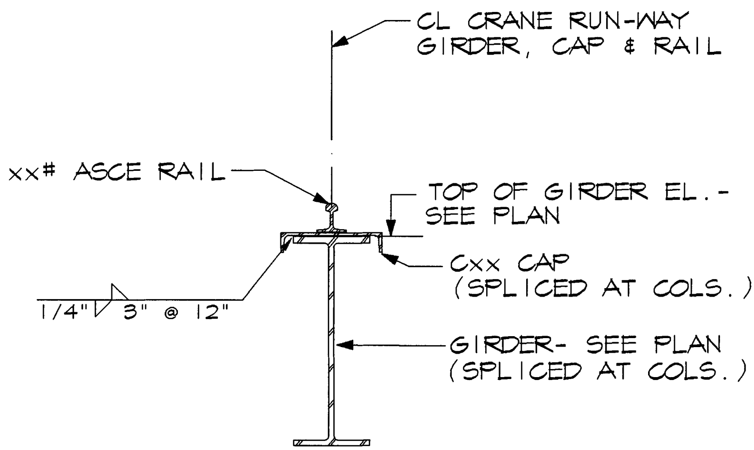
BRACED BAY DETAILS

NOT TO SCALE (DETAIL T5–BAYD5)



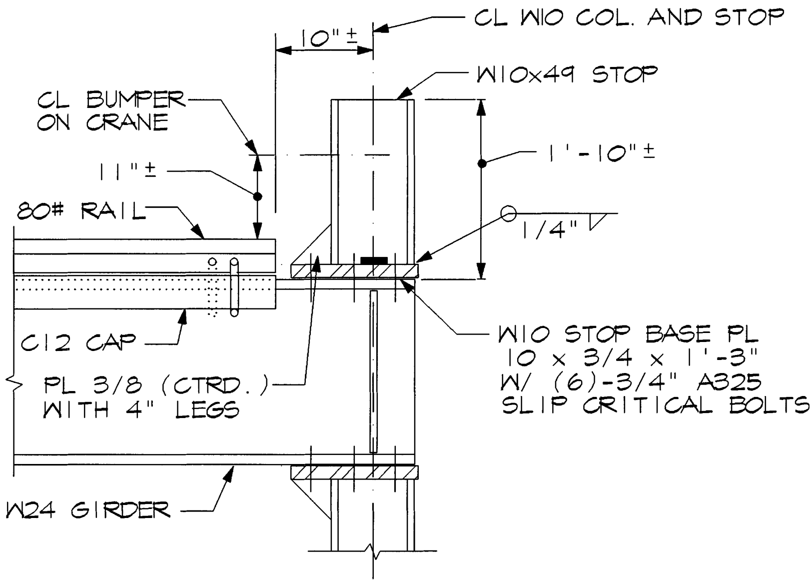
TYPICAL CRANE RAIL CONNECTION

NOT TO SCALE

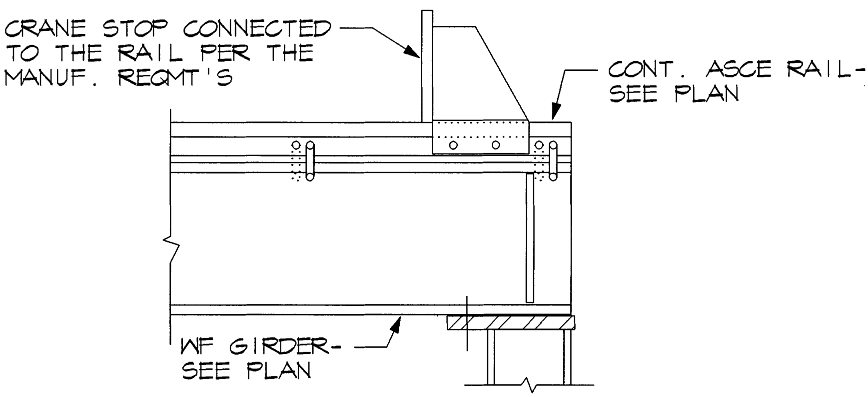


TYPICAL RUN WAY GIRDER DETAIL

NOT TO SCALE (DETAIL T5–CRAN1)



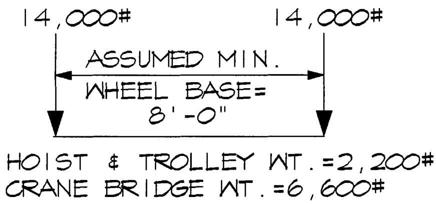
HIGH CAPACITY CRANE (30 TON MAX.)



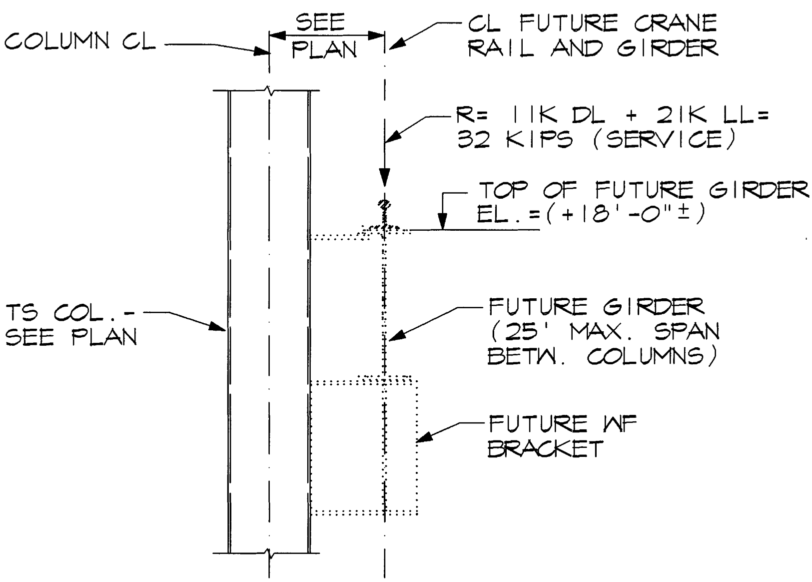
LOW CAPACITY CRANE (5 TON MAX.)

TYPICAL CRANE STOP DETAILS

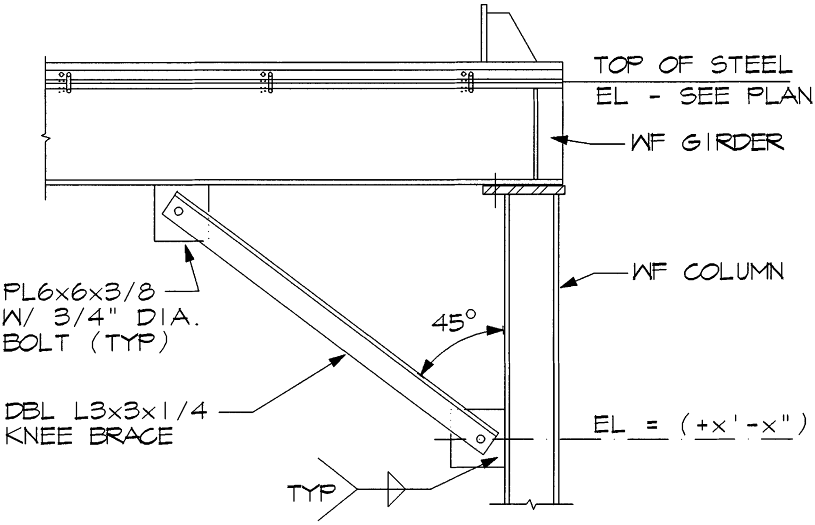
NOT TO SCALE (DETAIL T5–CRAN4)



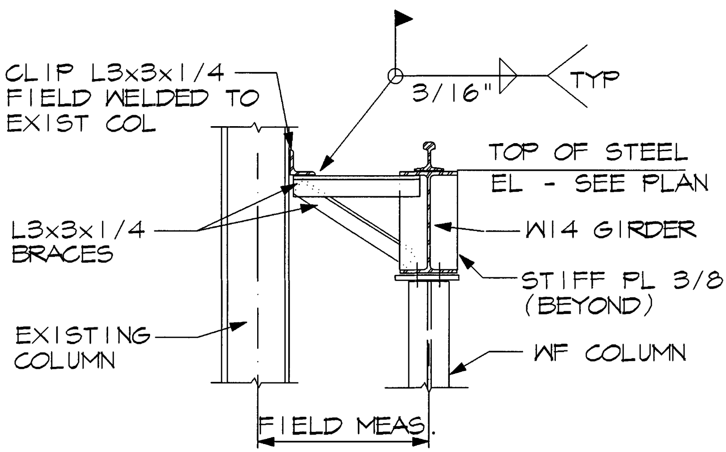
ASSUMED CRANE WHEEL LOADINGS
(2 TON CRANE, 2 WHEELS PER END TRUCK)



ASSUMED FUTURE CRANE CONDITION
NOT TO SCALE (DETAIL T5-CRAN8)



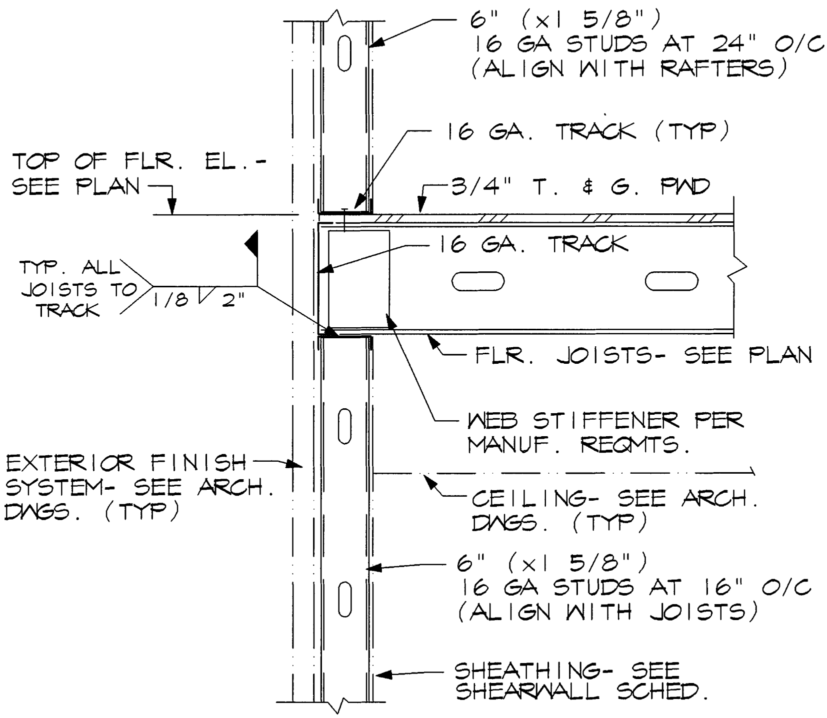
TYPICAL KNEE BRACE DETAIL



TYPICAL GIRDER BRACE DETAIL

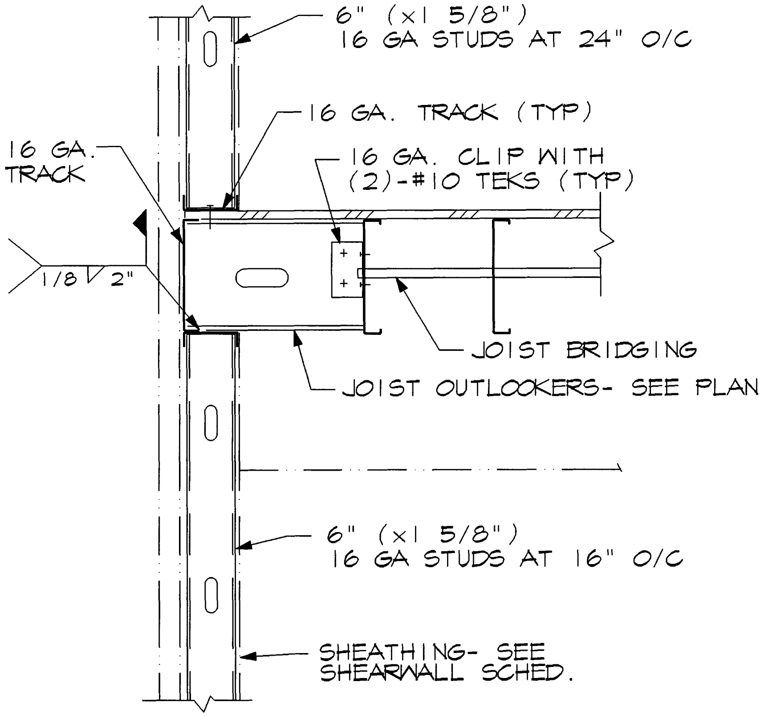
MISC. CRANE FRAMING DETAILS

NOT TO SCALE (DETAIL T5–CRAN9)



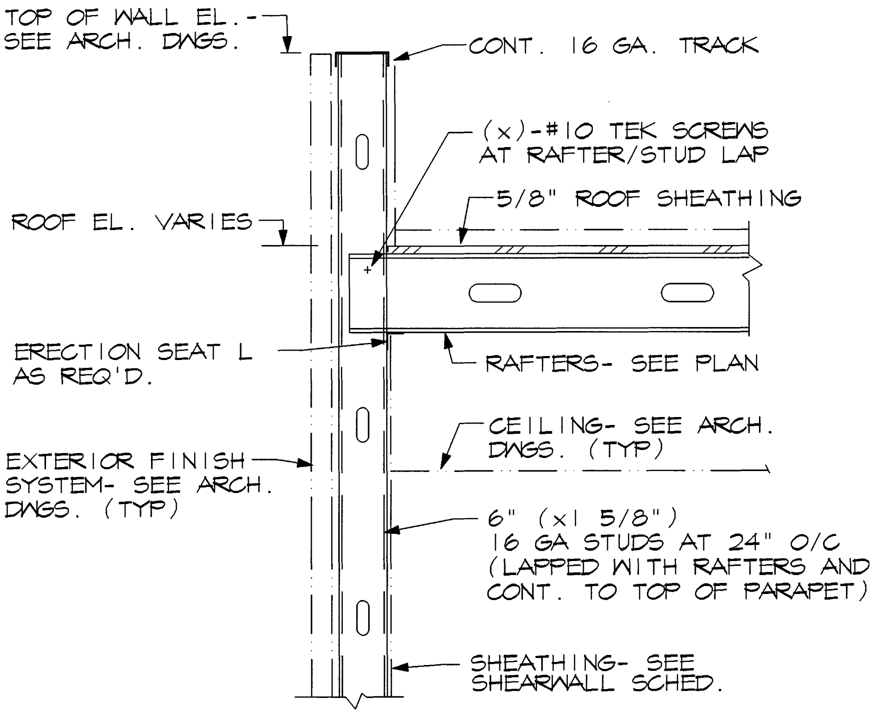
TYPICAL FLOOR JOIST BEARING DETAIL

NOT TO SCALE (DETAIL T5–CF1)



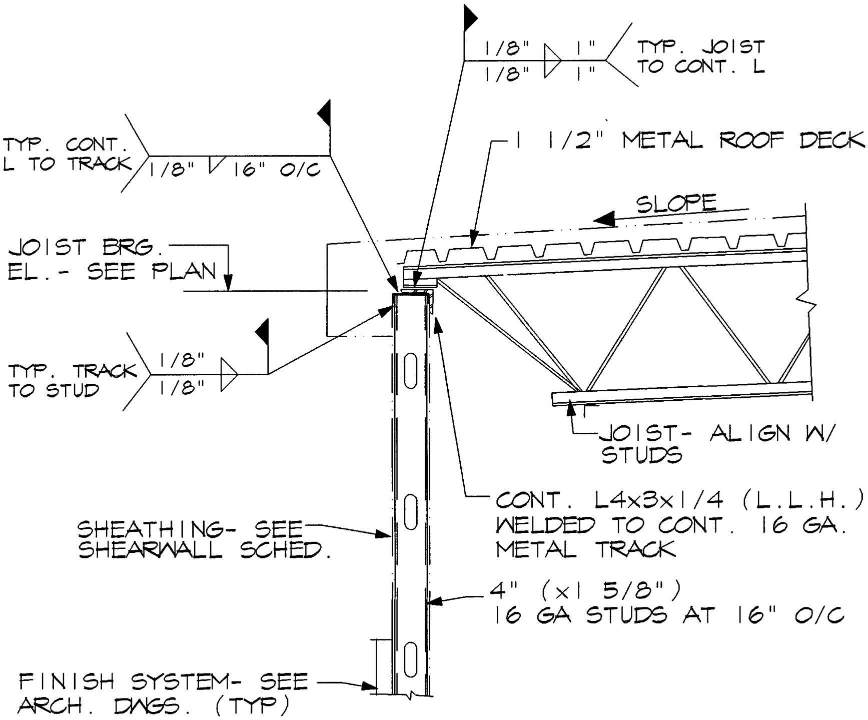
TYP. FLOOR JOIST NONBEARING DETAIL

NOT TO SCALE (DETAIL T5–CF2)



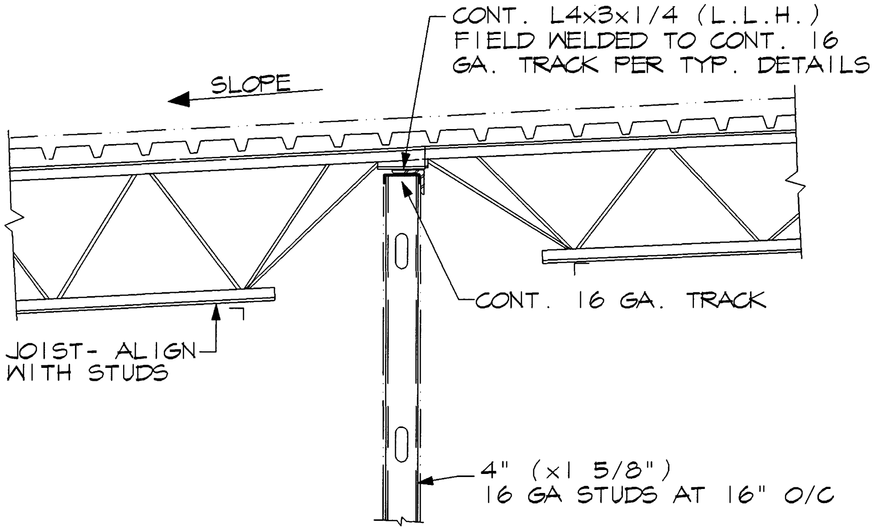
TYPICAL ROOF JOIST BEARING DETAIL

NOT TO SCALE (DETAIL T5–CF3)



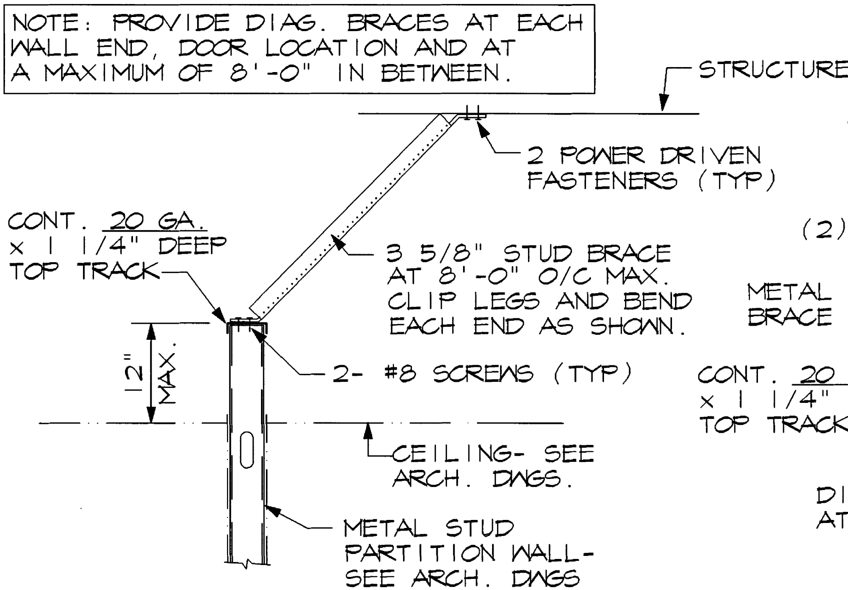
TYP. BAR JOIST ROOF BEARING DETAIL

NOT TO SCALE (DETAIL T5–CF7)

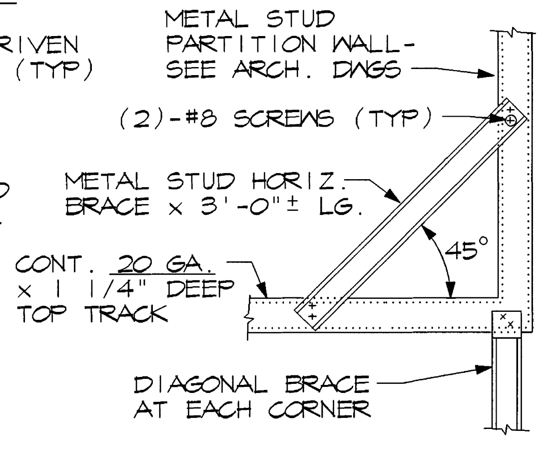


TYP. INTERIOR JOIST BEARING DETAIL

NOT TO SCALE (DETAIL T5–CF8)



ELEVATION – TYP. DIAGONAL BRACE

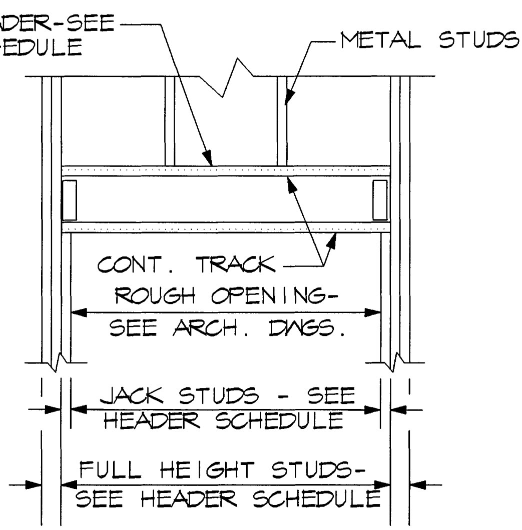


PLAN – TYP. DIAGONAL BRACE

TYPICAL PARTITION WALL BRACING DETAILS

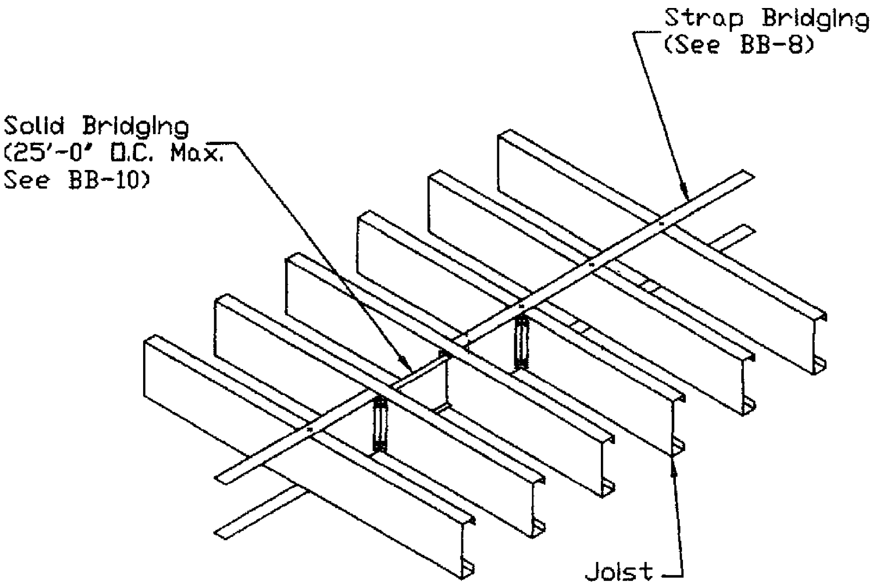
NOT TO SCALE (DETAIL T5–CFWAL)

COLD-FORMED METAL HEADER SCHEDULE					
MARK	SECTION	DESCRIPTION	GAUGE	JAMB STUDS	
				JACK	FULL-HT.
H-1		(2)- 6"(x1 5/8")	16 GA.	SINGLE	SINGLE
H-2		(2)- 8"(x1 5/8")	16 GA.	SINGLE	DOUBLE

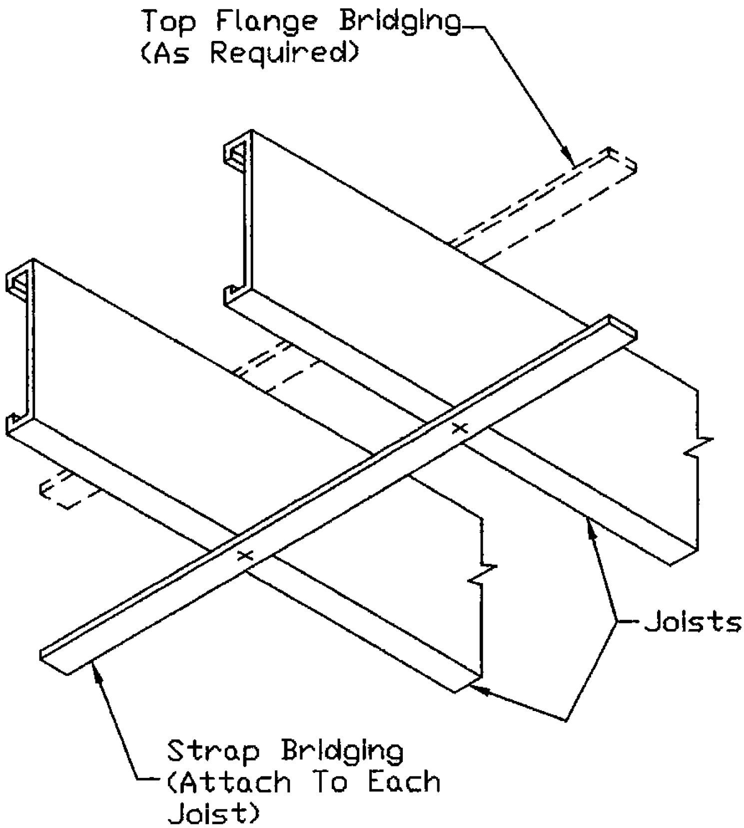


TYPICAL FRAMED OPENING DETAIL

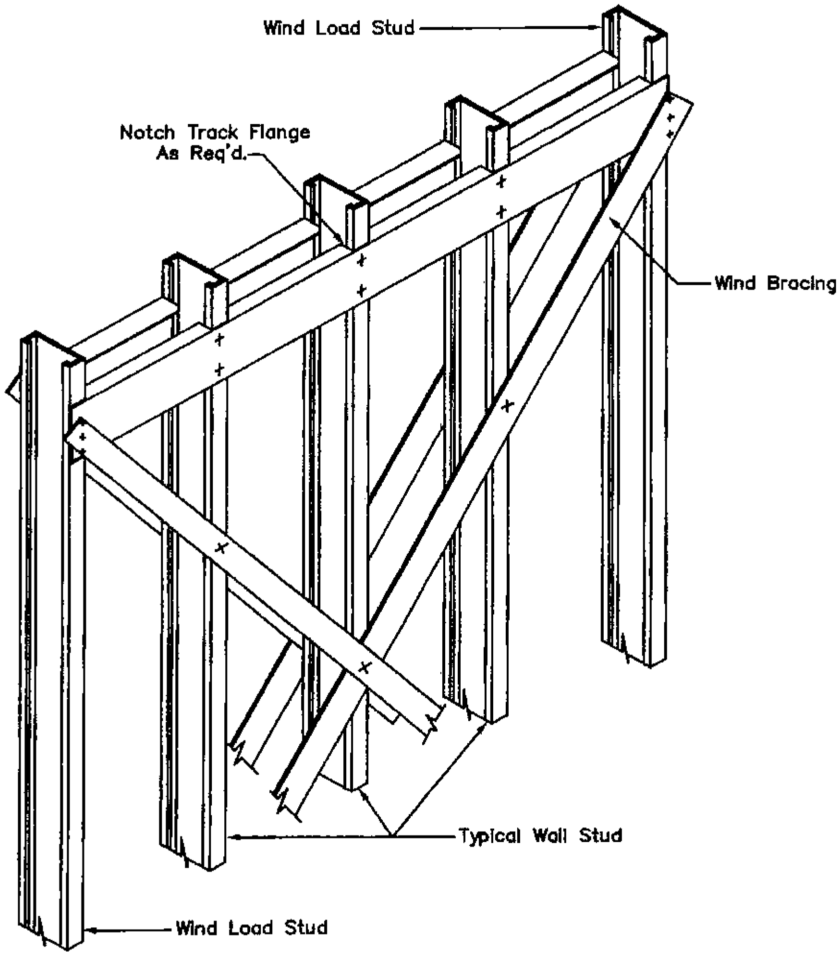
NOT TO SCALE (DETAIL T5-CFHDR)



BRIDGING AND BRACING – FLOOR BRIDGING

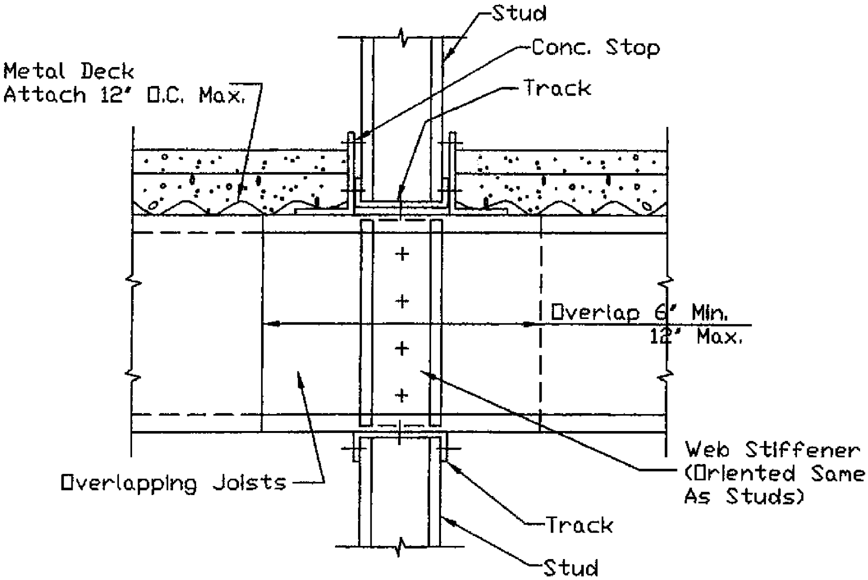


BRIDGING AND BRACING – STRAP BRIDGING



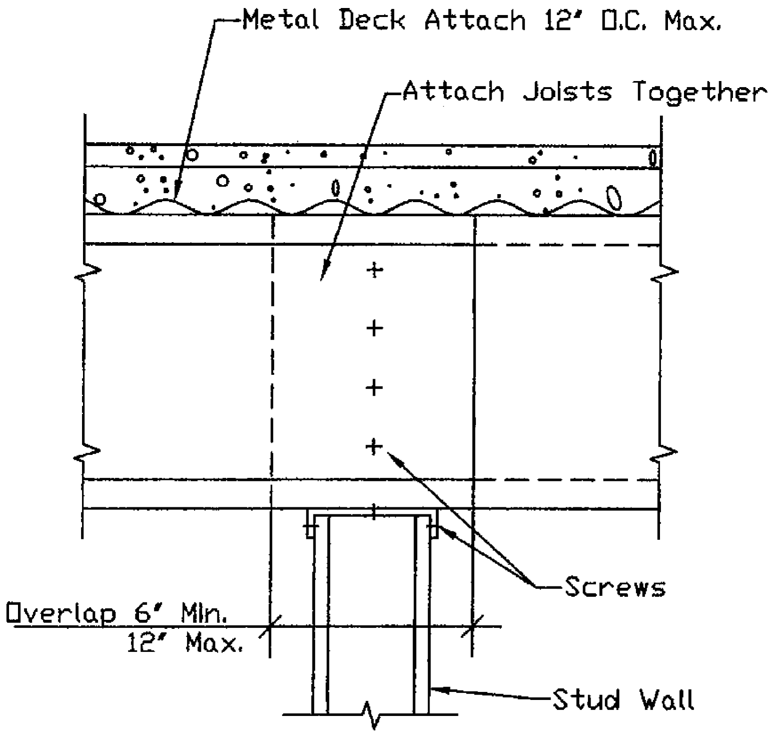
NOTE: NO. OF FASTENERS WILL VARY WITH STRENGTH REQUIRED

BRIDGING AND BRACING
Mid-Span Connection



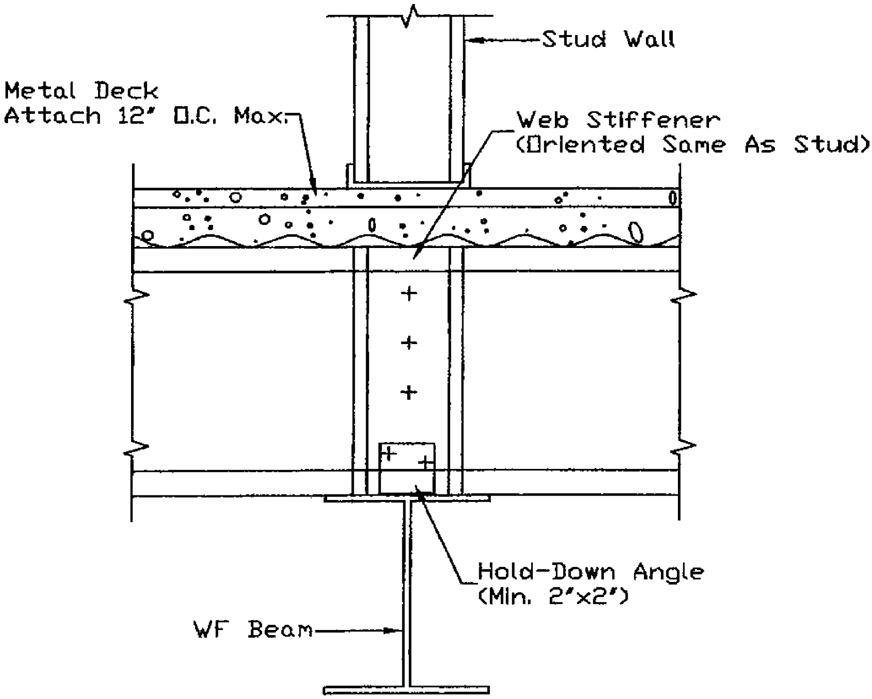
NOTE: ALIGN WEBS OF ALL MEMBERS

FLOOR SYSTEMS – CENTER BEARING ON STUDS



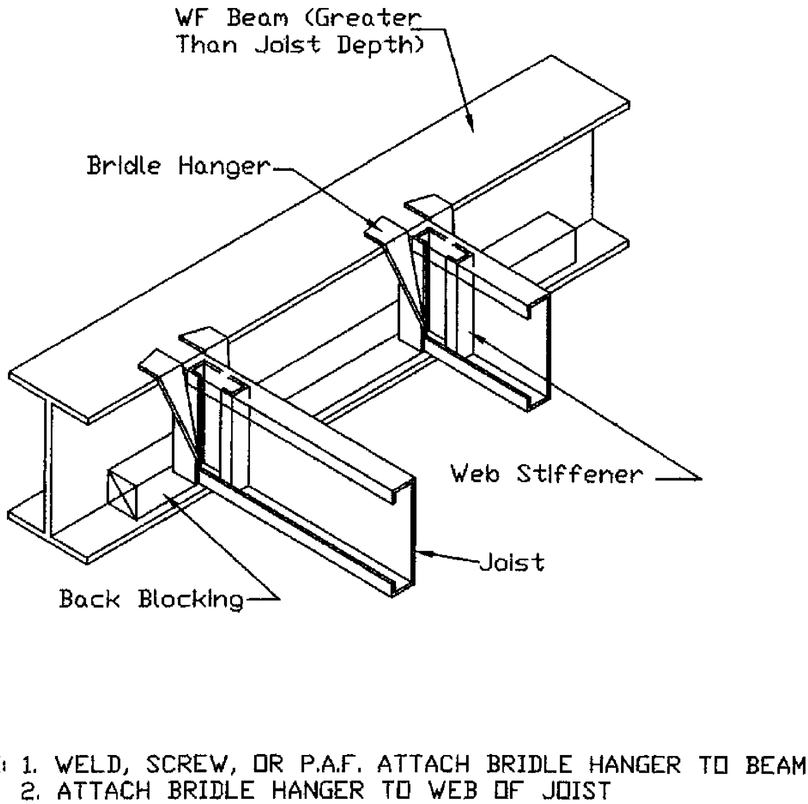
NOTE: 1. NO. OF SCREWS WILL VARY WITH DEPTH OF JOIST
2. ALIGN WEBS OF ALL MEMBERS
3. WEB STIFFENER MAY BE REQUIRED

FLOOR SYSTEMS - OVERLAPPING JOISTS



NOTE: 1. NO. OF SCREWS WILL VARY WITH DEPTH OF JOIST
2. ALIGN WEBS OF ALL MEMBERS

FLOOR SYSTEMS - BEARING ON STEEL BEAM



FLOOR SYSTEMS – CONNECTION TO WF BEAM

6.8 Reference

Williams, David R., *Structural Details Manual*, McGraw-Hill, 1998.

Chapter

7

Connection Design for Special Structures

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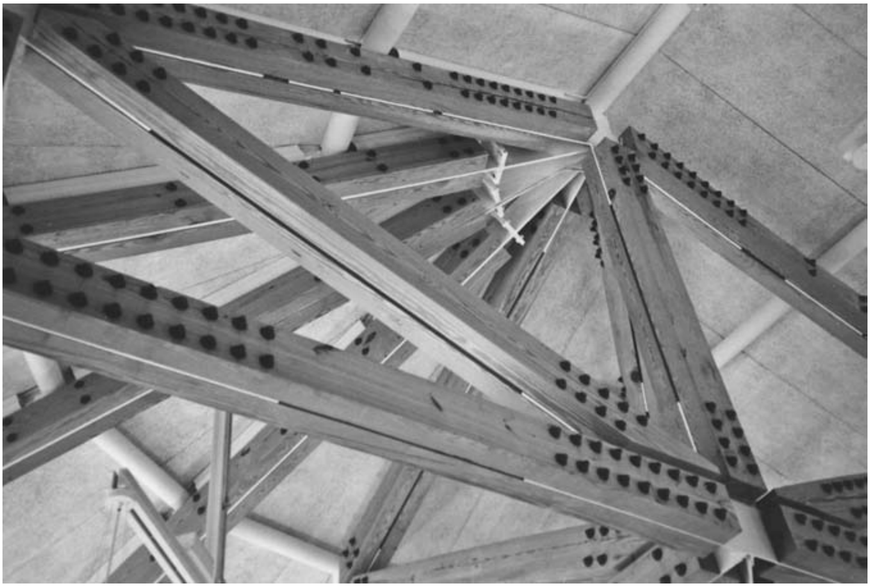
7.1 Introduction

Design drawings are intended to convey the engineer of record's (EOR) concept of the structure to the builder. As in any communication, there are always ample opportunities for misinterpretations or even a failure to communicate important information. The chance of a communication failure increases when constraints, such as time or budget, impact the drawing preparation and when the structural system involves unique, complex, or heavy members. Typically most of the engineer's design efforts involve laying out the structural system, structural analysis, and designing the members. Connections are often a last-minute addition to the drawing. They are usually communicated by use of schedules and standard details or, in the case of



(*Courtesy of The Steel Institute of New York.*)

unique connections, a representative detail. In a complex structure it is almost impossible for the designer's details to show the variations required to accommodate all of the loads, member sizes, and geometry required for special connections.

Traditionally, the structural engineer establishes the strength and stiffness requirements for all connections on the design drawings along with the preferred method of force transfer. The fabricator's engineer/detailer then develops connections that comply with these guidelines. The scope of this work may vary from only establishing detail dimensions to selecting the type of connection and sizing the connection material. When designing connections for special structures it is often necessary to develop connections that involve nonstandard details or at least a modification of standard details. It is important to clearly show load-transfer requirements and to work with the fabricator to design connections that can be economically fabricated and erected and still meet all structural requirements.

Unique connections for structures, such as long-span truss connections, space-frame connections, heavy-plate connections, and splices in group IV and V shapes, present special problems. Standard connections have been refined over the years and the problems are known. Every time you develop connections for new systems you have to be on the alert for unforeseen problems. Long-span truss connections must carry large forces while allowing for mill and fabrication tolerances and still provide easy assembly in the field. Heavy connections may have material and structural compatibility problems. Space-frame connections have access and dimensional tolerance problems. All of these may involve economic and constructibility issues that require input from the fabricator and erector.

The completion of structural design drawings marks the close of only part of the total connection design process when designing special structures. Connections often need to be modified for reasons of constructibility and economy during the detailing phase. With special connections the need for modification may even arise during the fabrication or erection phases of construction when constructibility problems are discovered. Special connections as mentioned previously have not been subjected to the trials of repeated use and unforeseen problems sometimes occur during construction.

Converting these design drawings into a structure requires a partnership between the EOR and the fabricator. Each has a role, the EOR as the designer and the fabricator as the builder. While they may assist each other, they remain solely responsible for the separate duties. The fabricator may size connections and propose changes in details and material but this is done as a builder not a designer. The engineer may help with construction by providing dimensional information or checking construction loads for the erector but the fabricator/erector remains responsible for the fit and constructibility of the structure. This design-build approach to developing connections works well for special structures.

The EOR may elect to show representative connections, the type of load transfer that is needed, along with the required connection design forces rather than attempting to dimension each connection. The fabricator then sizes the connection material based on these requirements and provides all of the detail dimensioning. The EOR can then review and verify that the connections are adequate for his or her design. This method of developing connection details utilizes the knowledge and experience of both the EOR and the fabricator in the most efficient way.

The detailing phase should start with a predetail conference with the EOR, fabricator, and where necessary, the erector and general contractor or construction manager attending. Preliminary sketches and schedules of some connections may be submitted at this time. This initial meeting is an effort by the entire construction team to understand the structural concept, verify whether all the needed information is shown, and determine if there are any obvious constructibility problems.

Connection design does not even stop with the approval of shop drawings by the EOR. The beginning of shop fabrication presents additional challenges. Material ordered for the project may not conform to specifications, fabrication errors may occur, and unforeseen constructibility problems might be discovered. The fabricator must evaluate each problem to determine if a modification or repair is necessary. Even though shop supervision or quality control personnel may identify the problem, it is important that the fabricator's engineer review and document any modification. If it is determined that the connection, even after repair or modification, will not meet the original standards, the proposed action must be submitted to the EOR to determine if the connection as fabricated will be fit for its intended purpose.

The erection of the steel frame serves as a check of the fabricator's efforts to detail and fabricate connections that fit perfectly. If the erector cannot put the bolt in the hole, it may be necessary to modify the connection. Most minor fit-up problems can be resolved with reaming, slotting, or shimming. Larger-dimensional errors or other constructibility problems may require the fabricator to develop a new connection detail that requires the approval of the engineer. Again, it may not be feasible to provide a connection that meets the original design standard and the EOR will be called on to make a fitness for purpose determination.

It is very important that the EOR be made aware of, and carefully review, any connection modifications during the entire detailing, fabrication, and erection of special structures. The load transfer is often so complex that only the EOR can evaluate the effect of any modification on the service and strength of the structure.

7.2 Lateral Load Systems

Bracing systems usually involve some of the most complex shop details, require the most labor to fabricate, and are the members most likely to have field fit-up problems. These members, however, are often shown with the least detail on the design drawing. Typical bracing elevations in addition to members, sizes and the location of the work point should show the complete load path with all of the forces. The connection designer must be able to determine how the loads accumulate and are transferred from the origin of the force to the foundation in order to design all of the connections for the appropriate forces. This includes knowing diaphragm shears and chord forces, collector forces, and pass-through forces at bracing connections. The designer's failure to provide a complete load path may result in critical connections not being able to deliver the design forces to the bracing system.

Diaphragm chords and drag struts often serve as gravity load members in addition to being part of the bracing system. It is important to design the connections for these members for both shear and axial load. Wide-flange beams with heavy framing angles can transfer axial force of the amount found in drag struts for most bracing systems. See Chap. 2 of this book for more detail on how to design this type of connection and the limits on capacity while still maintaining a flexible-type connection. When these struts are joist or joist girders, special connection details are required. Joist and joist girder end connections typically are able to transmit only a few kips because of the eccentricity between the seated end connection and the axis of the top chord. A field-welded tie consisting of a plate or pair of angles near the neutral axis of the top chord angles is the preferred method of passing axial forces across these types of joints. Drag struts with very large transfer forces occur when it becomes necessary to transfer the entire horizontal force of a brace to a brace in a nearby bay. Members with large axial forces will usually require heavy connections that will be rigid. Consideration should be given to designing these members with fully restrained connections.

The use of concentric work points at bracing joints makes the analysis of the frame and the design of members easier but may subject the connection material to eccentric loads or result in awkward, uneconomical details. This usually occurs when bracing slopes are extreme or member sizes vary substantially in size. It is important when designing bracing to determine if the work points chosen will result in efficient use of connection material. The connection of the diagonal bracing member to the strut beam and column can be efficiently designed using procedures such as the uniform force method found in Chap. 2 (see Fig. 7.1).

Shear-wall systems are simpler to detail and normally involve knowing only drag strut forces or diaphragm forces. These force transfer systems must be clearly shown and detailed. When the structure depends on shear walls, precast panels, or horizontal diaphragms for lateral stability, it is important that the general contractor and erector know this. The erector, by standard practice, provides erection bracing only for lateral loads on the bare frame. The construction sequence may require the general contractor or erector to provide additional bracing because the permanent lateral load system is not complete.

Moment frame systems are often very conservatively shown with notes calling for connections with a strength equal to the full section.

Figure 7.1 Large gusset plate due to concentric work point for brace.

While this may be required in seismic zones, lateral load systems are often designed for wind and the members are often sized for stiffness. There can be a substantial savings if the connections are sized for the actual design forces rather than the member strength. It is important when designing connections for wind frames to know the size of the moment in each direction. Typically, the maximum tension at the bottom flange is substantially less than at the top flange due to the combination of wind and gravity moments. When designing moment connections and checking column stiffener requirements, the use of these reduced tension loads may provide simpler, more economical connections.

7.3 Long-Span Trusses

Long-span trusses can be divided into three general types based on the methods of fabrication and erection. Trusses up to approximately 16 ft deep and 100 ft in length can be shop-fabricated and shipped to the field in one piece. When these trusses are over 100 ft, they can be shop-assembled in sections and shipped to the field in sections for assembly and erection. Trusses over 16 ft deep are generally fabricated and shipped as individual members for assembly and erection in the field. The first two types usually have standard connections that are discussed in other chapters. The third type, because of the size and loads carried, has special connection design requirements.

Deep long-span trusses typically use wide-flange shapes with all of the flanges in the same plane as the truss. If all of the members are approximately the same depth, connections can be made using gusset plates that lap both sides at the panel joints. When designing web members, it is important to look at the actual depth of the chord member rather than the nominal depth. For example, when using a W 14 $\times$ 311 for a chord where the actual depth is more than 17 in, it would be better to use a W 16 $\times$ 67 than a W 14 $\times$ 61 for a web member. The W 14 $\times$ 61 would require the use of $2\frac{1}{8}$-in fills on each side. Truss panels should be approximately square for the most efficient layout of gusset-plate connections when using Pratt-type configurations. When using Warren-style panel, it may help to increase the slope of the web member to make a more compact joint while reducing the force and length of the compression diagonal. Chords should, where possible, splice at panel points in order to use the gusset plates and bolts already there as part of the splice material. This also makes it possible to provide for camber or roof slope by allowing a change in alignment at a braced point. The gusset plates on a Pratt-type truss will be extended on the diagonal side to allow for bolt placement in this member. For this reason, the gusset plates should be first sized to accommodate the web member connections and the chord splice placed near the center of the plate rather than at the actual panel point intersection. The plate size is then checked for chord splice requirements.

This type of truss typically uses high-strength bolts for all connections. Traditionally, these have been bearing-type connections in standard holes. This requires either very precise computer numerical controlled (CNC) drilling or full shop assembly with reaming or drilling from the solid. Even with current CNC equipment it is very difficult with heavy members to obtain the tolerances needed for reliable field fit up when using standard $\frac{1}{16}$-in oversize holes. Shop assembly and reaming or drilling from solid is very expensive and because of the overall truss size it may not be possible for some fabricators to do this. There has been a trend in recent years to use oversize holes and slip-critical bolts to allow tolerances that are readily achievable by most drill lines. While this increases the number of bolts and gusset-plate sizes, this can be offset by using larger A490 bolts. Bolt material costs are approximately proportional to the strength provided. For example, while the cost of a 1-in-diameter A490 bolt is more than a $\frac{7}{8}$-in A325, the number of bolts required is substantially less. While the cost of the bolt material required does not change as the size and grade increase, the cost of plate, hole making, and installation costs decrease so larger-diameter higher-strength bolts are usually cost-effective.

Bolt size selection is also dependent upon the magnitude of force to be transmitted, the net section requirements of the members, and the tightening methods to be used. Generally for the loads and member sizes used in this type of truss, a 1-in-diameter A490 bolt is an efficient choice. The AISC specification provisions that use yield on the gross section and fracture on the net section generally make it possible to use oversize holes for most members without increasing section size. The exception to this may be chord splices where double gage lines are sometimes used. In this case it may be necessary to use two or three rows of bolts at a single gage as lead-in bolts (see Fig. 7.2). It is also important to check for shear lag using the net section provisions of the AISC specification when connecting only to the flanges of members. If possible, all bolts should be designed for single shear. This is especially true at splices that change slope since any splice plate on the inside will have to accommodate the change in slope by skewing the holes in a relatively narrow width plate. It may, however, be necessary to use bolts in double shear at tension chord splices to limit splice length. Compression chord splices should generally be designed as finished to bear type joints with bolts sized for half of the design force. Since these bolts will be slip-critical in oversize holes, it

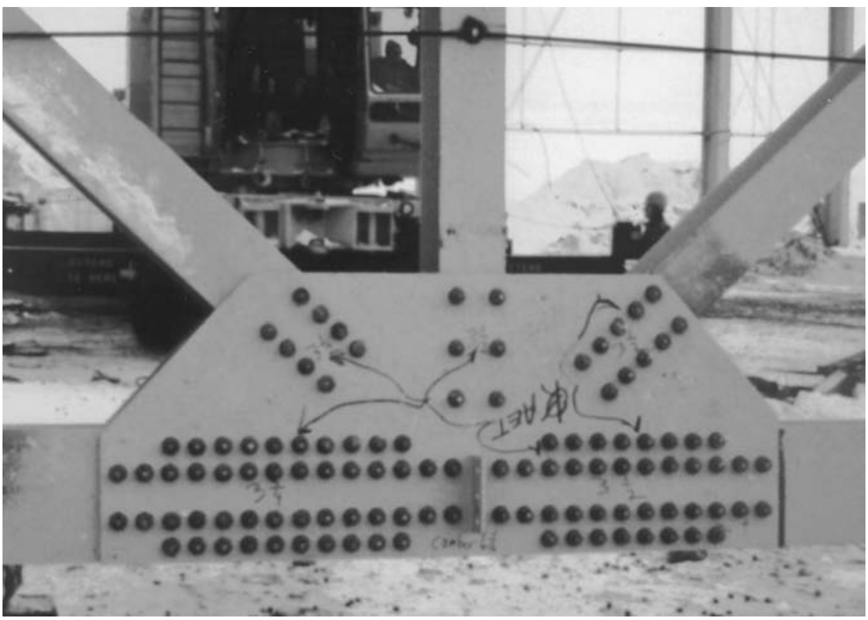


Figure 7.2 Large truss gusset plate with lead-in bolts at splice.

may require the use of mild steel shims in the joint to achieve the detailed chord dimension. Oversize holes should generally be detailed in all plies of material. This will allow the use of full-size drift pins to fair up the hole and make it easier to align the truss. Slip-critical bolts require special procedures to properly tension the bolts and must be inspected to ensure the required tension is achieved. While a slip into bearing is a service failure and not a collapse, it is important to establish a quality program that will ensure the work meets the design requirements.

Since most trusses will be assembled in large sections on the ground, it is important to design the major gusset-plate connections so all of the bolts, except the splices between sections, can be tensioned and inspected on the ground where they are easier to install and inspect. Secondary framing connections should be made with plates shop-welded to the gusset plates rather than using some of the truss connection bolts for both connections (see Fig. 7.3).

A trial joint should be assembled, tensioned, and inspected with the fabricator, erector, bolt supplier, independent testing laboratory, and the engineer of record present. Written procedures for both bolt installation and inspection for the project should be developed and agreed upon by all parties (see Fig. 7.4).

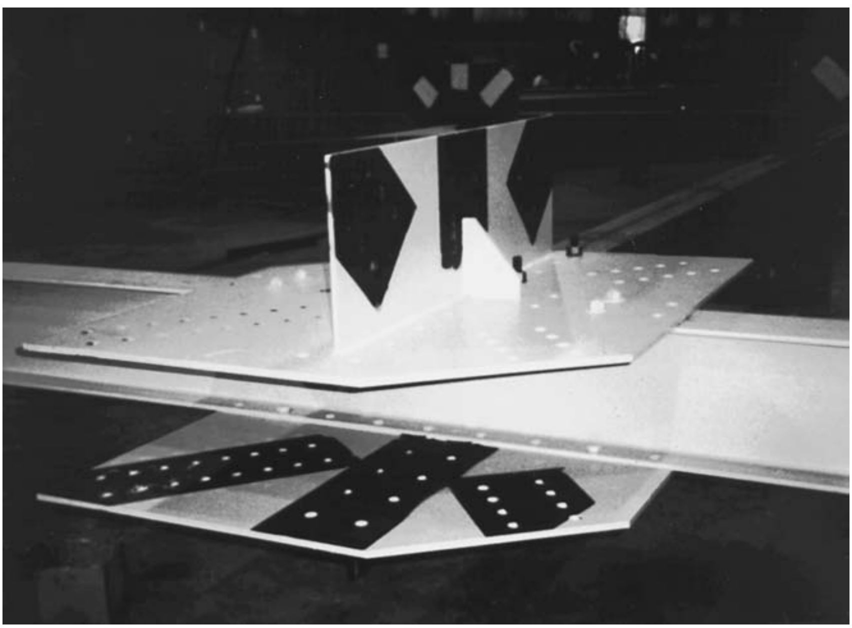


Figure 7.3 Truss gusset plate designed with bolts independent of secondary framing connections.

Figure 7.4 Trial joint assembly to establish bolt tensioning and inspection procedures—NWA Hangar, S. E. McClier and Computerized Structural Design.

7.4 Space-Frame Structures

The space truss form is often selected for either architectural appearance or because of depth limitations. Since one-way long-span trusses are easier to fabricate and erect, they will almost always be more economical than space trusses even though they will weigh more.

Space-frame structures have connections that must transfer forces on all three axes. They have access, dimensional tolerance, and through-thickness strength problems. Because of the complexity of these joints, it is important to try and develop some type of universal connector that can be reliably fabricated or to design the structure with large shop-welded assemblies that can be connected in the field. There are patented space frames that use special steel connectors. These are typically lighter structures with somewhat limited configurations which are not covered here.

Connectors for field assembly of structural steel space frames can be designed using a through plate for the major chord force and intersecting plates for members in the other planes. These intersecting plates are generally complete-joint-penetration–welded to the primary plate. When the geometry is especially complex, it may be necessary to use a center connection piece, usually a round member, to provide access to weld the joint. In either case, the through-thickness strains

Figure 7.5 Space-frame connector—Carver-Hawkeye Arena, S. E. Geiger—Berger.

Figure 7.6 Space-frame connector in welding fixture—Carver-Hawkeye Arena.

due to welding make it advisable to use a low-sulfur steel with a good through-thickness ductility. This material is expensive and not readily available so it is important to standardize on as few plate thicknesses as possible and use this only where needed. The welding procedure and filler metal should be evaluated to determine if it is adequate for both the design and fabrication requirements (see Figs. 7.5 and 7.6). The attachment of the truss members to these connectors in the field can either be by welding or bolting. Shop-welding and field-bolting may provide better quality control but this system generally requires two connections. Field-welding typically requires only one connection and generally provides more fit-up tolerance. If field-welding is used, it is important to try to use primarily fillet welds and, if possible, limit the out-of-position welding. Because some type of erection connection or shoring will be required until the structural weld is made, space trusses should, where possible, be designed so they can be ground-assembled. Their inherent stiffness allows them to be hoisted or jacked into final position after full assembly (see Fig. 7.7). This is very important for economy, quality, and safety.

Space trusses can also be designed so they can be shop-welded into panels of a size that can be shipped, thereby reducing the number of field connections. Shipping limitations will normally limit these panels to about 15 ft deep and about the same width. This size will allow the shop to rotate the panel and position it for efficient welding (see Fig. 7.8).

Figure 7.7 Space-frame module 42 by 126 ft being lifted into place—Carver-Hawkeye Arena.

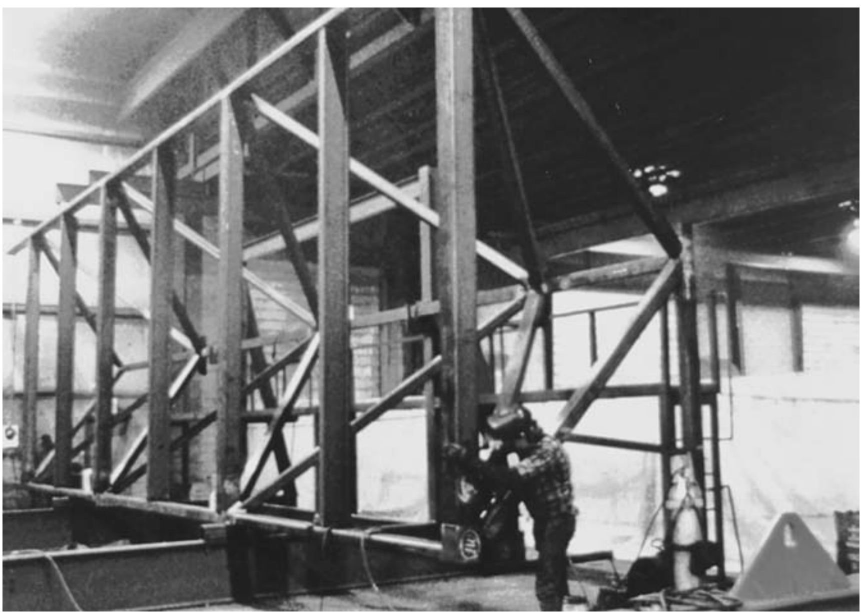


Figure 7.8 Space-frame module 15 by 60 ft rotated for shop welding—Minneapolis Convention Center, S. E. Skilling, Ward, Magnusson and Barkshire.

Figure 7.9 Solid 6-in^2 reinforcement for HSS joint—Minneapolis Convention Center.

Hollow structural sections (HSS) are often used for truss members because of their appearance and axial-load capacity. Connections of direct-welded HSS require special design procedures. The connection limit state can be various modes of wall failure in addition to weld rupture due to stress concentrations. These stress concentrations are caused by the difference in the relative flexibility of the chord wall when compared to the axial stiffness of the web member. The chord wall thickness required for connections is an important factor when designing members. It may be necessary to increase wall thickness or insert a heavy section at the branch to transmit the design forces. (See Fig. 7.9.)

Welds for HSS-to-HSS connections should be sized to ensure adequate ductility to prevent rupture at design loads. This can be easily accomplished using the effective length concepts given in the AISC *Hollow Structural Sections Connections Manual*[4] or the AWS/*Structural Welding Code* (ANSI/AWS D1.1-98).[5] A more conservative procedure would be to use a weld with an effective throat 1.1 times the wall thickness of the web or branch member. This is intended to make sure the wall of the web member will yield and redistribute stress before the weld ruptures. The ratio is based on E70XX electrodes and A500 grade B material. Direct-welded HSS connections of the T, Y, and K type should, where possible, utilize fillet or partial-penetration welds. Unbacked complete-joint-penetration welds that must be made from one side require special welder certification and are very difficult to make and inspect. Butt splices in HSS may require complete-joint-penetration welds. This type of weld should be made using steel backing to allow the use of standard weld procedures and welder certifications. For more detailed information on these connections, see Refs. 1, 3, 4, and 5.

7.5 Examples of Connections for Special Structures

Examples of connections developed for special structures can be helpful to illustrate the types of problems that are encountered and some idea of how connections can be adapted to meet special requirements.

The first project is a 42-story office building that uses a perimeter moment frame coupled with a braced core as the lateral load-resisting system. When a free-body diagram of the connection forces for the brace members in the core was prepared prior to developing connections, it was discovered that the axial loads for the horizontal struts given in the connection schedule were substantially less than the horizontal component of the brace diagonal. The EOR reviewed the lateral load analysis and discovered that when the structure was modeled, a stiffness factor was assigned to the floor diaphragm to provide for the interaction between the moment frame and the braced core. In the model the floor was carrying part of the brace force. While the brace loads may actually be transmitted in this manner, the EOR decided to follow conventional practice and size the steel for the full brace force rather than rely on this type of composite action. All of the horizontal struts were resized and connections were then developed for the full horizontal component of the diagonal force. Diagonal braces were wide-flange sections using claw angle-type connections with 1-in-diameter A325 SC bolts in oversize holes.

The second project is a sports arena using a skewed chord space truss supported on eight columns with the roof located at the bottom chord of the trusses (see Fig. 7.10). Each type of connection was clearly shown on the design drawing along with the forces to be used to determine the number of bolts and welds required for each connection. While reviewing the forces given for the bottom chord, it was noted that the bottom chord members had been modeled as axially loaded pin-ended members with vertical end reactions due to the roof dead and live loads on the bottom chord. A check of the actual connection which consisted of a plate on each side of the web that was welded between the flanges of W 27 sections indicated the connection was rigid. After reviewing this compatibility concern, the EOR decided to

Figure 7.10 Bottom chord connection for space truss—Carver-Hawkeye Arena.

size the connections for the axial forces and vertical end reactions given using *N*-type values for all of the bolts. A check of the connections using *X*-type values for the bolts indicated adequate reserve strength for possible end moments.

The exposed top chord, diagonals, and connectors were all made of ASTM A588 material left unpainted so it could weather. While the fabricator was detailing these connections, the EOR became aware of a study[2] that indicated, under certain conditions where moisture had access to the inside of a joint, the expansive pressure of the continuing corrosion could overstress the bolts and lead to failure. Connection details were modified to make sure the recommendations on minimum plate thickness and maximum bolt spacing were complied with. Special restrictive fabrication tolerances were established for connection material flatness in order to ensure the connection bolts would be able to clamp the full surface together. The fabricator, by using techniques such as prebending plate prior to welding and using heat-straightening after welding, was able to eliminate almost all distortion due to shop welds (see Fig. 7.11). The high-strength bolts were able to pull the plates together so there were no gaps in the connections (see Fig. 7.12).

The third project is a 57-story office building that uses a unique lateral load system. The wind in the longitudinal direction is resisted by

Figure 7.11 Heat straightening of connection plates after welding—Carver-Hawkeye Arena.

Figure 7.12 Exposed top chord connection showing fit-up after bolting—Carver-Hawkeye Arena.

Figure 7.13 Vierendeel framing system—Norwest Financial Center, S. E. CBM Inc.

five-story bands of vierendeel trusses that span 97 ft between concrete super columns. The vierendeel trusses were designed as horizontal tree girders with verticals spliced at midheight between floors (see Fig. 7.13).

These splice connections were first designed with partial-joint-penetration field welds. The shop connections of these W 24 verticals to the horizontal girder were complete-joint-penetration welds. The combination of weld shrinkage due to these shop welds along with the distortion of the girder due to welding and the rolling tolerance of the vertical section made it almost impossible to achieve the proper fit-up of the field-welded joint without a lot of expensive remedial work. Since the field splice was at the inflection point of the vertical, there were only axial loads and shears to be transmitted through the connection. It was decided to use an end-plate–type connection with slip-critical bolts in oversize holes to accommodate the fabrication and rolling tolerances. In addition, the members were detailed short and a $\frac{3}{8}$-in shim pack was provided to bring each joint to the proper elevation. The modification of this connection was one of the keys to the early completion of the erection of this structure (see Figs. 7.14 and 7.15).

The fourth project is a 37-story mixed-use structure that uses a megatruss bracing system for wind loads. The bracing truss has nodes at five-story intervals and uses wide-flange members for

Figure 7.14 Shimming of field splice of vierendeel verticals—Norwest Financial Center.

Figure 7.15 Splice of vierendeel verticals showing alignment—Norwest Financial Center.

stiffness. The connections of the truss at the nodes were designed as partial-joint-penetration groove-welded butt splices. Because of past experience with poor fit-up the EOR specified that joint fit-up had to comply with AWS D1.1 prequalified joint requirements with no build-out permitted.

The combination of mill, fabrication, and erection tolerances would have made it impossible to achieve this type of fit on these heavy W 14 members. It was decided to add a field splice in all of the diagonals midway between nodes using a lap-plate–type splice. This allowed the erector to position the lower half tight to the node and then jack the upper half tight to the upper node. The brace members had all been sized for axial stiffness and the design forces were typically less than half of the member capacity. The lap plates were designed and fillet welded for the actual brace force (see Figs. 7.16 and 7.17).

The fifth project is an exhibition hall consisting of three lamella domes 210 ft in diameter surrounded by a 60-ft-wide delta-type space truss made of hollow structural sections (see Fig. 7.18). Each dome is supported by a series of sloping pipe struts from four columns. The domes vertically support the inside of the space truss and the space truss laterally constrains the domes (see Fig. 7.19). The total

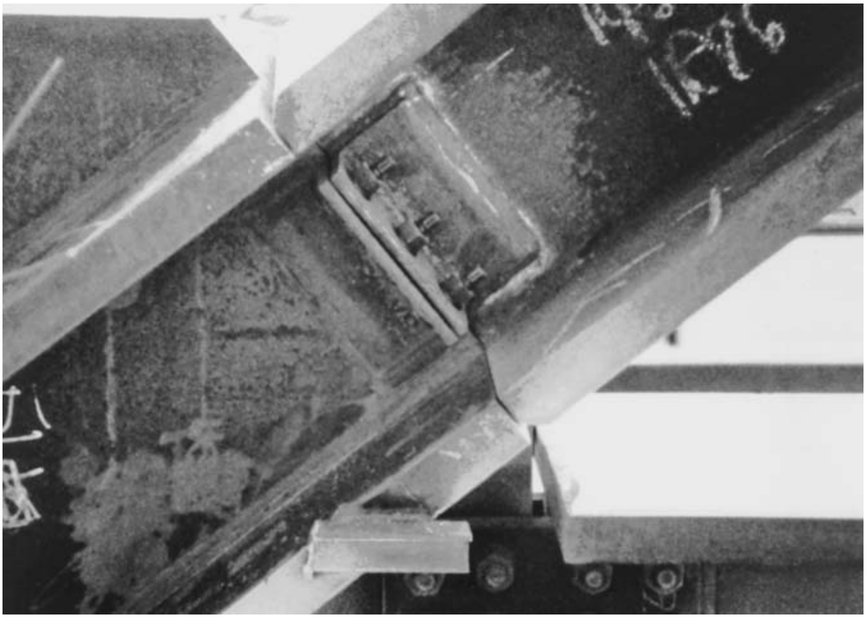


Figure 7.16 Bracing node connection showing fit-up—Plaza Seven, S. E. CBM Inc.

Figure 7.17 Adjustable midheight splice of bracing diagonal—Plaza Seven.

Figure 7.18 Lamella dome and delta space frame—Minneapolis Convention Center.

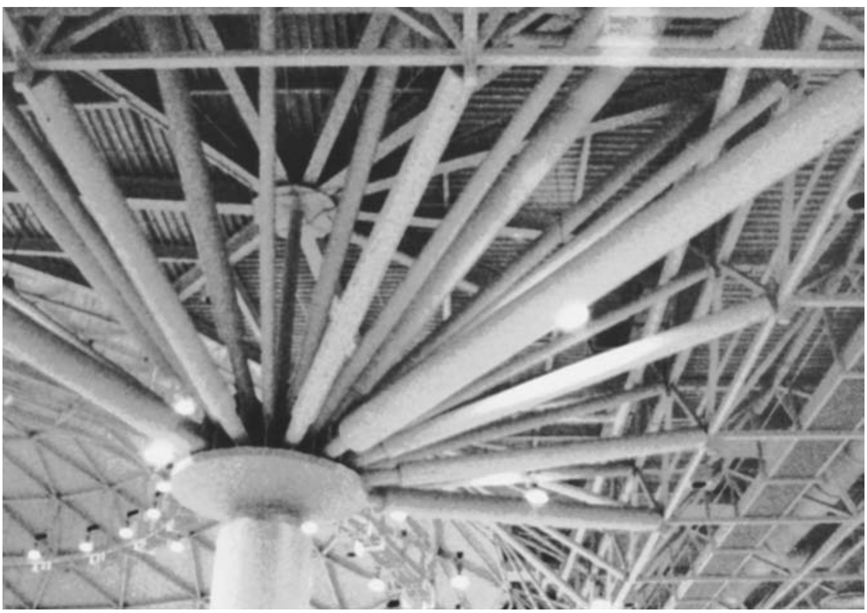


Figure 7.19 Dome and space-frame column and pipe supports—Minneapolis Convention Center.

Figure 7.20 Top-chord field splice of delta space frame—Minneapolis Convention Center.

structure is approximately 900 ft long without an expansion joint. The EOR laid out the space truss so modular units could be shop-fabricated in units 15 ft wide and 60 ft long. The top and bottom chords of these units were offset and oriented in the 60-ft direction to minimize splicing (see Fig. 7.20). Each unit had two top chords and one bottom chord. This resulted in double top chords at the splice between units. These chords were connected by flare V-groove field welds at the panel points. The bottom chords were detailed with a short connector stub to which a section of cross-chord was butt-welded in the field.

The diagonals of the delta truss intersect the bottom chords at 45° to the vertical and the chords. These members were typically 6-in-square HSS and would have overlapped at the panel point. To avoid this the EOR detailed a connector consisting of intersecting vertical plates on top of each chord. Initially it was planned to provide complete-joint-penetration welds for these diagonal connections. However, when weld procedures were developed, it became apparent that the restricted access to these joints would make both welding and inspection very difficult. The connection was redesigned using partial-joint-penetration and fillet welds sized for the actual loads in the members with allowances as required for uneven load distribution (see Figs. 7.21 and 7.22).

All of the butt splices in the chord were detailed as complete-joint-penetration welds using internal steel backing so a standard V-groove weld could be used (see Fig. 7.23).

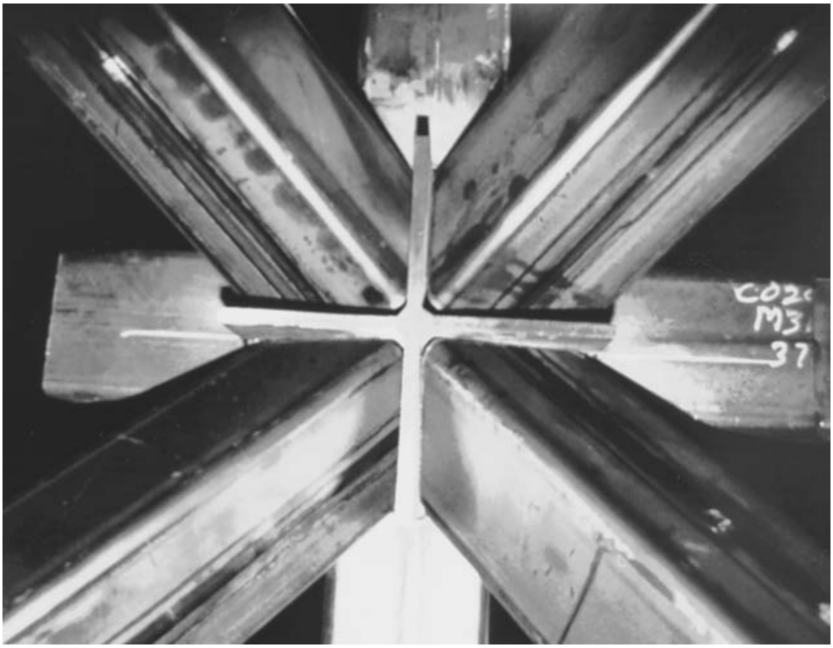


Figure 7.21 Space-frame bottom-chord connection showing fit-up—Minneapolis Convention Center.

Figure 7.22 Space-frame bottom-chord showing weld—Minneapolis Convention Center.

The connection of the sloping pipes to bottom ring of the dome consisted of a series of radial plates that were complete-joint-penetration–welded to a 6-in-thick connector plate on the ring (see Fig. 7.24). The EOR was concerned about possible brittle fracture of these heavy welded plate connections and specified material ductility using

Figure 7.23 Space-frame bottom-chord splice connection—Minneapolis Convention Center.

Figure 7.24 Gusset-plate connections for pipe struts to dome—Minneapolis Convention Center.

standard Charpy V-notch testing. When orders were placed, the material supplier informed the fabricator that the standard longitudinal Charpy test would not measure the through-thickness properties needed to accommodate the weld strains. The design and construction team consulted with metallurgists and fracture mechanics experts to develop a specification and testing procedure that would ensure adequate through-thickness ductility. The testing procedure called for through-thickness samples to be taken near the center of the plate. A minimum through-thickness reduction in area of 20%, along with a minimum Charpy value of 15 ft-lb at 70°F in all three axes, was specified. While the through-thickness Charpy test is not a reliable indicator of ductility, it was decided to do this test as a general comparison with the properties in the other two directions. The producer supplied a low-sulfur, vacuum-degassed, and normalized material with inclusion shape control. All material was 100% ultrasonically inspected at the mill. There were no through-thickness problems due to welding strains. Since this project was built, several mills have developed proprietary low-sulfur materials with excellent through-thickness properties and ASTM now has a specification, A770, for through-thickness testing.

The lamella domes were designed using wide-flange shapes shop-welded into diamond patterns (see Fig. 7.25). Since the fabricator

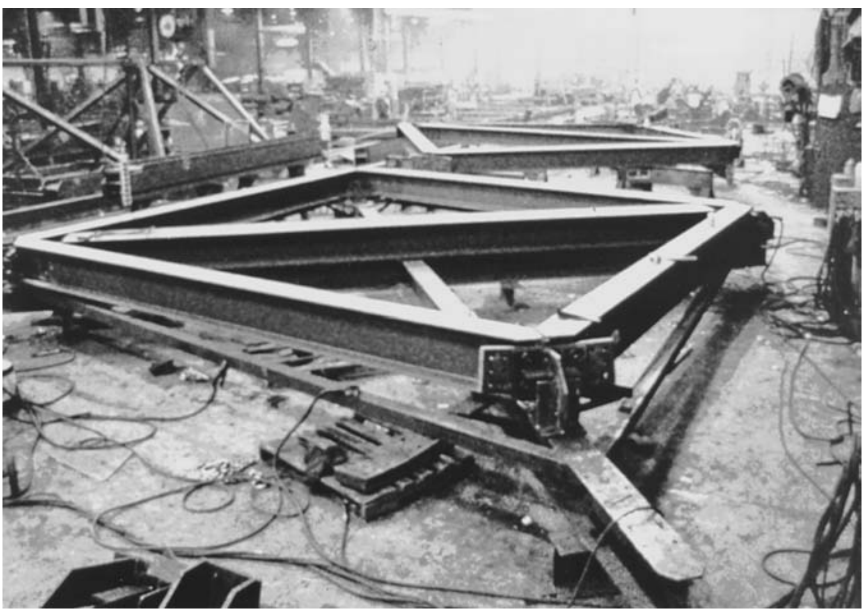


Figure 7.25 Lamella dome module in fabrication—Minneapolis Convention Center.

Figure 7.26 Start of ground assembly of dome—Minneapolis Convention Center.

was nearly even, the 24-ft-wide diamonds at bottom ring were shop-fabricated and delivered to the site. A bolted web splice was provided between diamonds and for the ring beams to the diamonds. This provided both an erection splice and was adequate for any out-of-plane loads. The flanges of these members were complete-joint-penetration–welded.

The entire space frame project was ground-assembled. The space trusses were assembled in units 60 by 75 ft and hoisted by crane onto shoring towers and perimeter columns. The dome was assembled ring by ring on the ground using shores as required (see Fig. 7.26). The dome assembly, including the deck, was then jacked into place (see Fig. 7.27). When the slotted pipe supports were slipped over and welded to the gusset plates on the heavy weldments described here, a new concern arose. The misalignment of the gusset plates due to the angular distortion caused by the one-sided groove welds along with the erection tolerances of the structure resulted in some bowing of the connection plates. The EOR reviewed the forces and added stiffeners, where required, to prevent buckling due to any misalignment of the plates in compression.

The sixth project is a multiuse sports and events center. The roof framing consists of 26-ft-deep trusses spanning 206 ft that

Figure 7.27 Jacking rods in position for lifting dome—Minneapolis Convention Center.

are framed at one end to a jack truss spanning 185 ft. All of the truss members are W 14 sections oriented with the flanges in the vertical plane and connected with lap-type gusset plates on each flange. All of the typical truss connections used slip-critical high-strength bolts in oversized holes. Chords were spliced at panel points and W 16 sections were used for web members as recommended previously.

The connection of the 206-ft trusses to the jack truss presented a special problem because of the large reaction that had to be carried by the framing angles (see Fig. 7.28). Originally it was planned to intersect the work points of the end connection at the center of the jack truss top chord. A free-body diagram of the connection, however, showed the bolts would have to develop a moment of 465 ft-kips in addition to carrying a shear of 450 kips. Even if it was possible to get enough bolts in the plate, the angles could not develop the moment. By moving the work point to the face of the truss it was possible to eliminate the bending in the outstanding leg of the angle and to reduce the eccentricity to the bolt group in the other leg to 4.5 in. The eccentric reaction on the jack truss was easily balanced by adding a 14-kip axial connection at the bottom chord. The bending stress in the jack truss vertical was checked and found to be acceptable.

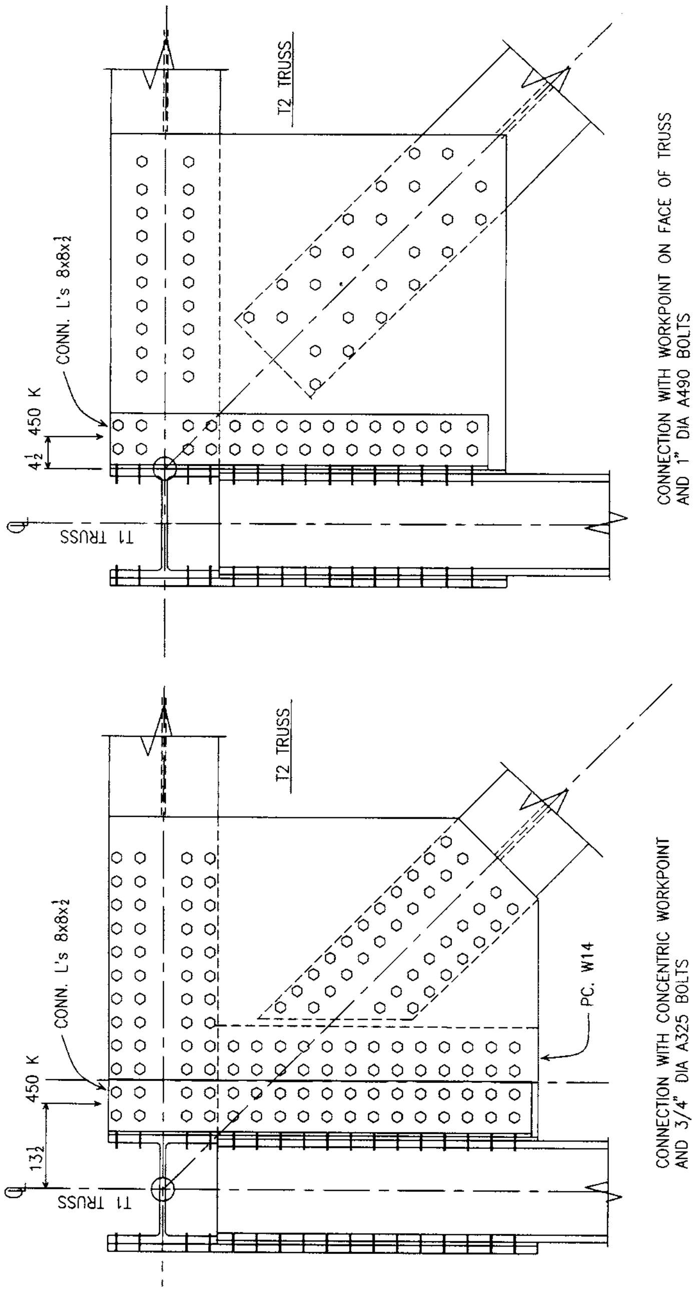


Figure 7.28 Connection roof truss to jack truss—Mankato Civic Center, S. E. MBJ.

7.6 Building Information Model (BIM)

The acronym, BIM, describes what has become a paradigm shift in the method of designing and constructing complex steel structures. Historically the design and construction of a building has relied upon drawings and specifications as described in Sec. 7.1 to define a building. Multiple views were required to depict an object often along with some type of written description of the size and other requirements. This process also requires each contractor to interpret the data and reproduce much of the same information before adding their product. Coordination between various systems such as concrete, structural steel, precast, and mechanical systems required time and a great deal of effort and was subject to error.

A BIM differs from a two-dimensional (2D) and some three-dimensional (3D) CAD models, which are typically nothing more than electronic drawings. Instead of representing members as lines with labels, a BIM is an intelligent model that consists of a series of objects with their geometry and attributes. These objects or building elements can be displayed in multiple views, as well as having their nongraphic attributes assigned to them. If the particular BIM software has interoperability with other programs, the various design and construction team members with the help of other softwares can use the BIM information to develop their own model and their work can be input back to the BIM. This provides for ease and accuracy in coordinating the design and construction of the various building elements.

There are several levels or versions of BIM. When it involves steel fabrication and erection Mark Howland, Chief Engineer of Paxton Vierling Steel uses the terms "Big BIM" and "Little BIM." A *Big BIM* is where the design team models the structure and uses it for their design needs and then passes it to the entire construction team for their use and input. This method is most cost-effective from the overall project standpoint. It allows the designer to better visualize each of the elements and avoid clashes between various systems. Doing this up front in the design stage saves both time and money. Design firms have expressed concern about the cost and possible legal liability for providing this information. Owners need to realize there is a substantial value in providing this service up front and should compensate the designers accordingly. A *Little BIM* is where the design team provides 2D drawings or CAD files and one member of the construction team, typically the steel fabricator prepares a BIM model for steel fabrication and erection. This model may in turn be used by other members of the construction team to coordinate their work.

Two examples where the model was developed by the design team are the Adaptive Reuse of Soldier Field in Chicago, Illinois, and the New York Mets Stadium—Citi Field in Queens, New York.

The Soldier Field project involved an existing stadium built in the 1920s that featured classical colonnades designed by architect Holabird and Roche as a memorial to the American Soldier. Because this was an existing facility currently in use, the construction schedule was limited to 20 months. Thornton-Tomasetti, the structural engineer, elected to perform the structural analysis for the main frame using SAP2000 and RAM for the floor framing. Thornton-Tomasetti in joint decision with the project team proceeded to model the project in Tekla Corporation's Xsteel 3D-modeling software. The 3D models were generated for each of the four quadrants and documentation was added showing beams sizes, forces, and camber along with column and brace information.

The steel fabricator, Hirchfeld Steel Co. received the 3D models and used them for connection-design information and preparation of shop drawings. This model with the connection information added was then submitted for review by Thornton-Tomasetti. Because the review process only required the examination of the connections the review took 5 days instead of the usual 10 days and saved valuable schedule time. After the review process Hirchfeld Steel Co. used the information in the model to prepare computer numeric control (CNC) instructions for download to the machines to cut, punch, and drill material along with the preparation of 2D shop drawings. The model was also used to coordinate the detailing and erection of the secondary framing for the complex cladding system. A more complete description of the project can be in seen in Ref. 6.

The New York Mets Stadium—Citi Field is another example where the design team prepared the BIM for use by the construction team. WSP Cantor Seinuk decided to use AutoDesk's Revit software for this project. The project architect, HOK Sport, took full advantage of the 3D-modeling features of Revit along with its compatibility with other AutoDesk products. The architectural and structural models were combined to provide coordination of the complex geometry and construction features. At this stage a decision was made to convert the Revit BIM model to Tekla Corporation's Xsteel to facilitate the steel-detailing process. As a result of an AISC initiative, several years ago the industry established a digital standard for electronic communication, CIMsteel, Integration Standards/Version 2 (CIS/2). This provided interoperability between these software systems. What was unusual for this project was that the Xsteel program was furnished to the bidders to save them time and expense in bidding the project. The final Xsteel model was furnished to the selected steel supplier thereby saving considerable time and expense in preparing shop drawings and avoided numerous Requests for Information (RFIs). For a paper covering this project along with more information on BIM practices see Ref. 7.

LeJeune Steel Company has fabricated projects with what could be called Little BIMs with mixed results. The major problem with developing a BIM using 2D drawings is that depending on the accuracy of the data furnished by the contract documents it can be very time consuming often requiring numerous RFIs. While clashes or coordination issues are discovered before construction starts they still cause delays and added cost to resolve at this stage. A BIM developed for a project from conventional 2D drawings for the building structure and an AutoCad wire frame model for the facade framing found clashes between the facade framing and a number of building elements. While it was important to find these problems in the detailing stage it would have saved time and money to have found and solved the problems in the design stage.

The new University of Minnesota Football Stadium—TCF Field—utilized a somewhat different process to produce a BIM. The general contractor, M. A. Mortenson (MAM), worked with the design team to develop a model of the stadium geometry with all of the control points and their elevations using Nevis Works software. They supplied this information to the steel fabricator, LeJeune Steel Co. (LSC). LSC and its detailer, LTC Consultants, then built a Xsteel model with all of the structural steel members and added all of the connection details (see Fig. 7.29).

The interoperability between the two systems allowed all of the information to be transferred seamlessly. The interoperability also allowed LSC to subcontract a portion of the structural steel to

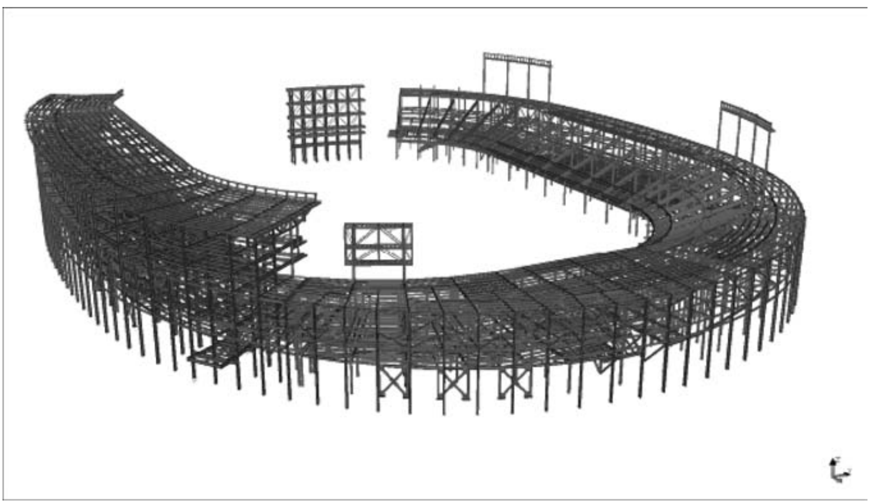


Figure 7.29 Model of the structural steel for the University of Minnesota, TCF Stadium, S. E. Magnusson—Klemencic Associates.

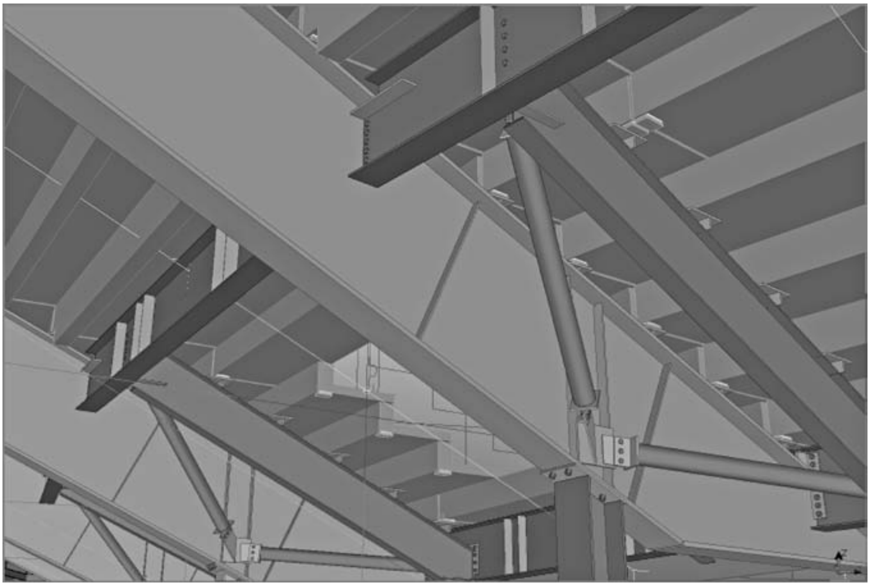


Figure 7.30 Model of steel rakers with precast seating—University of Minnesota, TCF Stadium.

American Structural Metals (ASM) who used the MAM information to build a Design Data SDS-2 model. LTC was able to import this information to check coordination between the steel packages and provide a full model of all of the steel to MAM.

MAM also built a model of the precast seating including the connection points to the steel. LSC incorporated this information into the Xsteel model and used it to determine the location of all of the seat support points on the steel seating rakers (see Fig. 7.30).

This not only saved time, it ensured that even with the complex radial geometry, the two materials would fit together without a problem. Initially there were some clashes but with all of the information available in one model, the problems were easily resolved in a few working sessions.

MAM also used the BIM to help with scheduling access to critical areas. There were areas under the cantilevered skyboxes where the precast erector and the steel erector had to take turns (see Fig. 7.31). The BIM allow the trades to see exactly what the access would be at various points in the construction process.

MAM also used the Xsteel model to coordinate the layout of the MEP duct work through the numerous brace elevations (see Fig. 7.32). This prevented clashes with the braces and the large gusset plates needed for their structural connections. Other subcontractors such as

Figure 7.31 Steel raker with precast seating erected—University of Minnesota, TCF Stadium.

Figure 7.32 Model showing bracing members and connections—University of Minnesota, TCF Stadium.

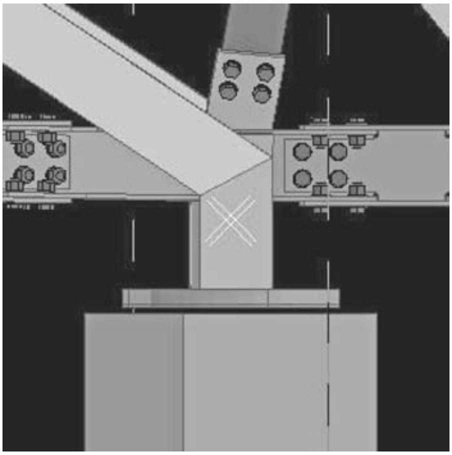


Figure 7.33 Front view of support node—Millennium Park, Chicago, Illinois, S. E. SOM.

the architectural precast contractor and even the window-washing equipment contractor used the model in laying out their work.

A BIM is really the only practical way to design, detail, fabricate, and erect complex structures like those described here. During the design stage the BIM not only allows the designer to check for clashes between various materials it also allows for an interactive viewing of each connection node to check for clearances needed to fabricate and erect the complex connections. The 3D node shown in Fig. 7.33 can be viewed through 360° rotation, both horizontally and vertically, along with section cuts anywhere on the assembly. The section cut shown in Fig. 7.34 shows the arrangements of the bolts at the connection and can be used to determine if the erection clearances are adequate to stick and tension the high strength bolts. The detail can be viewed over the Internet simultaneously by the structural engineer, fabricator, and steel detailer and any required modifications can be agreed upon. It is only necessary than to provide either an electronic or paper copy of the final design for record.

Most fabrication shops still require 2D shop-detail drawings to fit and weld the individual shipping pieces. Transferring all of the complex geometry can really only be done efficiently and accurately by detailing software. The rib member shown in Fig 7.35 illustrates some of the detail drawing complexity that is possible using a 3D model.

The ability to download all of the geometry from the model to the CNC machines that the fabricator uses to cut, punch, and drill all of the detail pieces not only provides significant cost and time savings, it provides a level of accuracy that greatly improves overall fabrication and erection accuracy. Experience has shown that fit of the detail parts often controls the accuracy of the shipping piece. When assembling a

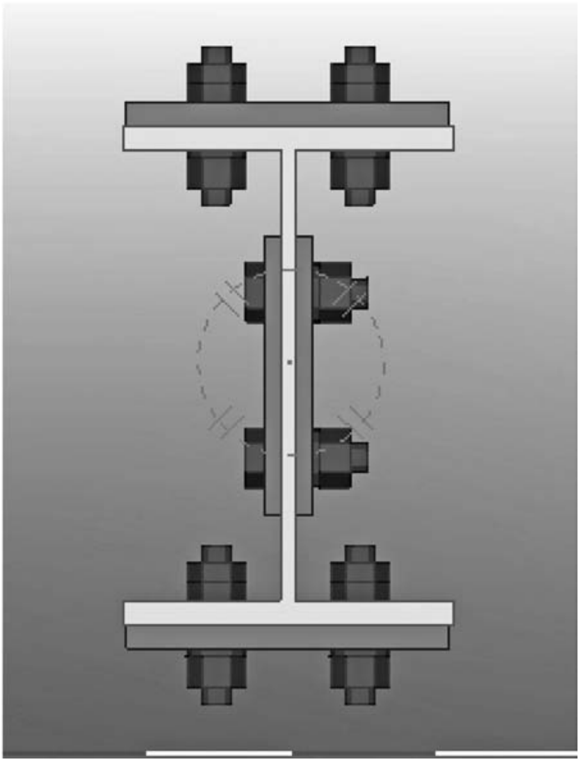


Figure 7.34 Section at support node—Millennium Park, Chicago, Illinois.

complex member if the parts do not fit the reason is not the accuracy of the parts but the alignment of the main pieces. When the alignment is adjusted so that the parts fit, the member typically will meet all planned dimensions.

The information from the BIM can also be used to build fixtures for complex assemblies and to design shoring for field erection. The model will provide coordinates for any point on the surface of the assembly along with the relationship to any other point needed to control overall fit. A fixture for shop assembly of a large erection subassembly is shown in Fig. 7.36. The shoring for the erection subassemblies shown in Fig. 7.37 was designed using the geometry from the BIM.

The BIM is often used in the erection process to verify access and plan the work sequence. For projects with free-form geometry like the Millennium Park project shown in Fig. 7.38 the correct positioning of each of the nodes can only be determined by using the coordinate geometry from the BIM. Coordinates are first determined for a series

Figure 7.35 Rib connection detail—Millennium Park, Chicago, Illinois.

Figure 7.36 Fixture for shop check of subassembly—Millennium Park, Chicago, Illinois.

Figure 7.37 Subassembly shoring—Millennium Park, Chicago, Illinois.

Figure 7.38 Using 3D coordinate system to locate steel—Millennium Park, Chicago, Illinois.

of targets placed on the surfae of the member near the nodes. This information is then downloaded to a total station–type surveying instrument. The total station with the target coordinate information is then used to verify the location, in space, of each node as a crane lifts the subassembly into place.

7.7 Conclusion

It is helpful when designing special connections to start with a free-body diagram of the connection. The free body should usually be cut at the connection face and all forces shown (see Fig. 7.39). While it may be necessary to use advanced techniques, such as finite-element analysis or yield-line theory to evaluate stiffness of elements, it is important to first try to establish the best force path. Care should be taken to make sure this is a complete path. Both sides of the connection must be able to transmit the force. An evaluation should be made for any connection eccentricity. It may be better to design the member for the eccentricity instead of the connection. A check should also be made for the flexibility of the connection if it was modeled as a pin in the analysis. It may be possible to ignore connection fixity in axially loaded members, such as trusses, as long as the members are modeled with flexible connections and all loads are applied at panel points. Preliminary connection design should be done prior to final member selection. It is impossible to effectively size members without taking into account connection requirements. Because of constructability and economic concerns, the design of special connections will almost always require input from the fabricator and erector.

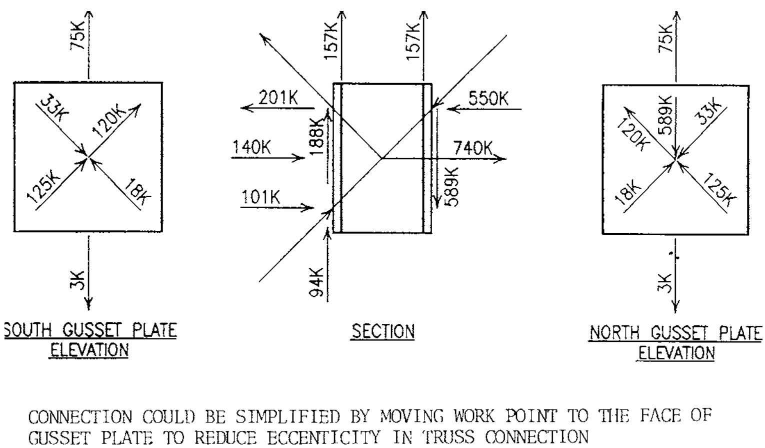


Figure 7.39 Free-body diagram of roof truss to box truss—NWA Hangar.

The EOR must either obtain this input in advance or be prepared to evaluate proposed means and methods modifications during the construction stage.

The increasing geometric complexity of special structures especially those with free-form geometry will require the use of new 3D solid–modeling programs to verify the constructability of the connections. It is important that the programs used have interoperability as outlined by the CIMsteel, Integration Standards/Version 2. This interoperability will allow for exchange of information and permit the fabricator to detail and download all of the required information for the CNC machines in his shop. This will help achieve the dimensional accuracy needed to make sure all of the members fit properly in the field.

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Chapter 8

Inspection and Quality Control

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(*Courtesy of The Steel Institute of New York.*)

8.1 Fastener Quality Control and Testing

The quality of the fastener components begins with the manufacturer of the steel. Steel is purchased by bolt, nut, and washer manufacturers to rigid compositional specifications so that, after manufacture and, if needed, heat treatment, the desired mechanical properties will be achieved. The quality of the steel is verified through the use of mill-test reports provided by the steel mill and reviewed by the fastener manufacturer. Treatment of the as-provided steel is sometimes necessary to prepare the steel for the manufacturing operations. This includes annealing, or softening, of the steel to make it more workable in the machinery. For bolts, drawing is needed to make the steel the exact diameter required.

The manufacturer will make several hundred to several hundred thousand components in each production lot, depending upon the type of product and the manufacturing facility. A *production lot* is defined as those items from the same lot of steel produced in the same equipment and heat-treated, lubricated (if added), and tested at the same time. Testing is performed during production to verify that dimensional tolerances are met. Physical testing is performed following the completion of heat treatment, if performed, and following any galvanizing application.

The type of testing required depends upon the type of product being manufactured. Test requirements are specified in the applicable ASTM specification. Bolts are tested for strength and ductility. Additional tests are required for A490 bolts and galvanized assemblies. Nuts are tested for stripping strength on a threaded mandrel, with a block attempting to push the nut off the end of the mandrel. Hardness tests are also performed to verify proper nut strength. Hardness tests are performed on washers.

Bolt strength is tested in a tensile testing machine. A wedge of either 6° or 10° is placed under the head of the bolt, then the bolt is pulled to failure. The failure must take place in the threads of the bolt, between the nut and the head. Failures directly underneath the bolt head, in the shank, or by stripping of the threads are unacceptable. The elongation of the bolt is also measured as tensile loading is applied. The bolt must satisfy the requirements for minimum proof load, which is established as 70% of the minimum specified tensile strength for A325 bolts and 80% of the minimum specified tensile strength for A490 bolts. The proof load establishes that the bolt will not yield prematurely at a low stress level, and therefore not provide the pretension desired when installed using established techniques. When bolts are too short to fit into a tensile testing machine, they are tested using hardness-test methods to verify minimum and maximum strength levels. All A490 bolts receive additional magnetic particle testing to detect microcracking.

Bolts that are galvanized must be supplied as an assembly, with the washers and nuts that are to be used with the bolts. A rotational-capacity test is performed to verify that the effect of galvanizing and the overtapping of the nut did not adversely affect the assembly performance. The test involves deliberately overtightening the assembly in a test fixture, ensuring the bolt and nut has adequate strength, then verifying that the threads of the bolt and nut resist stripping.

Special tests, also called *rotational-capacity tests,* are performed when fastener assemblies are to be used in bridge construction. This applies to both black and galvanized assemblies. The testing includes checking the torque requirements for tightening the assembly, with a maximum torque value used to confirm the effectiveness of the nut lubrication. The testing also ensures the fastener assemblies achieves required strength and overstrength, and verifies that the threads of the bolt and nut resist stripping.

Fastener components are physically tested using statistical sampling techniques, as prescribed by the applicable American Society for Testing and Materials (ASTM) specifications. Zero defects in strength and proof-load requirements are permitted. Should the item fail a strength, proof-load, or hardness test, if used, then the entire production lot is rejected. Generally, lots rejected on the basis of strength are heat-treated again, and then retested.

8.2 Bolt Preinstallation Inspection

Bolt inspection is a multistep operation that begins before the bolt installation starts. The following steps should be included in a bolt-inspection program:

1. Check the materials and certifications. Check that the certifications match with the supplied product. Verify that the materials supplied comply with the project specifications. Review the product certification papers for ASTM and project compliance.
2. Check for proper storage conditions. The various fastener components should be kept separate by production lot until time for installation. Preassembly of bolts, nuts, and washers prior to use by the installer is satisfactory as long as lot control is maintained. Proper storage also includes protection from the elements, maintaining adequate lubrication and keeping the materials free from dirt, sand, grit, and other foreign materials.
3. Check the assembly of bolt, nut, and washer (if used) in a bolt calibration device for material quality, verifying that it is capable of achieving the required pretension without breaking, thread stripping, or excessive installation effort. For bridge work, the American Association of State Highway and Transportation Officials (AASHTO) specification requires that two assemblies of each bolt lot, nut lot, and washer lot, as supplied and to be assembled and installed in the shop or field, be tested at the site of installation with a special rotational-capacity test. This test verifies material strength and ductility, thread stripping resistance, and the efficiency of the lubrication.
4. Check the validity of the installation technique for that group of fasteners. Perform the selected installation technique in a bolt tension calibration device, or with a "calibrated" direct tension indicator (dti) if the bolt is too short to fit into the calibrator. Verify that at least the minimum required pretension, plus 5%, is achieved using the specified technique. For the calibrated-wrench method, observe the calibration of the wrenches before the start of the work each day. This test is commonly combined with the test described in step (3).

5. Verify the knowledge of the installation crew. The previous two steps should be performed by the installation crew. By observing these tests, the crew demonstrates to the inspector their knowledge of the proper technique.
6. Check the faying surface condition of the fabricated steel for proper conditions prior to erection. This may include coating type and thickness, condition of curing, and cleanliness.
7. Check the size and quality of the bolt holes in the fabricated steel, including removal of burrs where necessary.

8.3 Bolt Installation and Inspection

The first step in installing bolts is the proper snugging of the joint. If the joint is not properly snugged, no pretensioning method will work correctly.

The majority of bolts in buildings need be tightened only to the "snug-tight" condition. Certain joints transferring load by shear/bearing must be pretensioned, as well as slip-critical joints and some direct tension joints.

The definition of *snug tight* is stated in the American Institute of Steel Construction (AISC) specification Section J3.1: "The snug-tight condition is defined as the tightness attained by either a few impacts of an impact wrench or the full effort of a worker with an ordinary spud wrench that brings the connected plies into firm contact."

If the joint is not in solid contact, the pretensioning method employed may fail to achieve the proper pretension for the bolts in the joint. Pretensioning the first bolt in the group will only serve to further draw down the gap between the steel elements. The installer erroneously assumes the first bolt is tight. The next bolt tightened further draws down any remaining gap, and the initial bolt becomes looser still. This can become a compounding series in some joints.

8.3.1 Turn-of-nut installation method

The turn-of-nut method has been around since the 1950s. The current "turns" table has been in use since 1978.

The principle behind the turn-of-nut method is the controlled elongation of the bolt. Because of the pitch of the threads, turning the nut a prescribed rotation elongates the bolt a certain amount. The elongation has a direct correlation to the bolt pretension. As bolts become larger in diameter, the number of threads per inch decreases accordingly; therefore, the same number of turns will provide at least the required amount of pretension for a given length-to-diameter ratio.

TABLE 8.1 RCSC Table 8.2, Turn-of-Nut Rotation

	Disposition of outer face of bolted parts		
Bolt length (underside of head to end of bolt)	Both faces normal to bolt axis	One face normal to bolt axis and other sloped not more than 1:20 (beveled washer not used)	Both faces sloped not more than 1:20 from normal to bolt axis (beveled washers not used)
Up to and including 4 diameters	1/3 turn	1/2 turn	2/3 turn
Over 4 diameters but not exceeding 8 diameters	1/2 turn	2/3 turn	5/6 turn
Over 8 diameters but not exceeding 12 diameters	2/3 turn	5/6 turn	1 turn

Table 8.1 provides the required turns for given bolt length-to-diameter ratios, as provided in the Research Council on Structural Connection (RCSC) specification Table 8.2. As an example, with flat surfaces and bolts less than or equal to 4 diameters in length, say a $^3/_4$- by 3-in bolt, a one-third turn must be provided. A $^3/_4$- by 5-in bolt would receive one-half turn. A $^3/_4$- by $6^1/_2$-in bolt would receive two-thirds turn.

For bolts over 12 diameters in length, too much variation exists to provide tabular values. It is required that the installer uses a bolt tension calibration device to determine the number of turns necessary to provide the required bolt pretension.

The sloping surfaces provisions apply when there is a slope to the surface beneath the bolt head or nut. This slope must not exceed 1:20, or approximately 3°. Extra rotation is needed to overcome the loss caused by the bending at the head or nut; therefore, a one-sixth turn is added for each sloping surface. If the slope exceeds 1:20, a beveled washer must be used equal to the slope.

If the sloping surfaces are caused by the $16^2/_3\%$ (10°) bevel used for channel and S-section flanges, then a standard $16^2/_3\%$ beveled washer must be used. The required turns increase for the sloping surface is not required, because the beveled washer has returned the head or nut to the parallel condition.

There is a tolerance to the amount of applied rotation. For turns of one-half or less, the nut may be under- or over-rotated by no more than 30°. For turns of two-thirds or more, the nut may be under- or over-rotated by no more than 45°. If the nut does not receive sufficient rotation, the desired pretension may not be achieved. The potential risk from over-rotation is that the bolt may be stretched to the point of breaking, or to the point where nut stripping may occur. However, fasteners that have exceeded the prescribed rotation plus tolerance are rarely cause for rejection should the fastener not break.

The installation sequence should start with snugging the joint. Following inspection for snug, the installation crew is permitted to matchmark the end of the bolt and a corner of the nut. The crew applies the required turns from Table 8.1 (Reproduced from RCSC Table 5) and the joint is inspected to verify the applied turns by checking the matchmark rotation.

The installation crew may also use the "watch the wrench chuck" method for turn-of-the-nut, electing not to matchmark. The inspector must monitor the crew's efforts to verify that the proper technique is routinely applied during the pretensioning.

8.3.2 Calibrated-wrench installation method

The calibrated-wrench method uses a special type impact wrench to tighten the bolts. The wrench is adjusted so that it stops impacting when the bolt has achieved at least the required pretension, as determined using a bolt tension calibration device, but does not overrotate the fastener beyond the turn-of-nut tables. Rather than impact until the wrench operator releases the trigger, the wrench automatically stops impacting when a certain resistance is felt by the wrench.

Pneumatic calibrated impact wrenches depend upon an internal cam unit for control. When the desired resistance, actually torque, is reached the cam unit shifts and the wrench stalls out. If the air pressure or air volume is inadequate, however, the control mechanism will not function properly and will continue to impact the fastener, although at a slower, weaker level. For this reason, the calibrated impact wrench must be calibrated with a given air-supply condition. The wrench should be calibrated using the same compressor and pressure settings, air hose, and air-hose length that will be used on the work. If an additional wrench is to be driven off the compressor, the wrench calibration should be checked with both wrenches in operation simultaneously as well as individually. If a significant length of hose from compressor to wrench is either added or removed, then the wrench should be recalibrated.

Calibration of the wrench is required every day, before installation begins, with three fastener assemblies of each diameter, length, grade, and lot. An assembly, by specification, would be comprised of a bolt from a specific production lot, a nut from a specific production lot, and a washer from a specific production lot.

If there is a significant difference in the quality of fastener lubrication, then the wrench must be calibrated for the varying lubrication conditions. A well-oiled bolt, washer, and nut assembly will require considerably less torque than one that is nearly dry or one that exhibits some indications of rust. Hence, if the wrench is calibrated using well-oiled fasteners, then used on a poorly lubricated fastener, the resultant bolt pretension will be less. The same concerns apply if the bolt, nut, or washer surface contains dirt, grit, or sand. Likewise, if a wrench is calibrated using a dry or rusty fastener, then used on a well-oiled fastener, the bolt may fail if the wrench fails to stall.

Efficient calibration becomes key to the use of the calibrated-wrench method of installation. Some projects have used a separate calibrated wrench for each given diameter, length, and grade being installed, changing wrenches when a different bolt group is being installed. Every wrench is calibrated each morning. It is possible to simplify operations by calibrating the wrench to properly install a wider range of bolts. This would be accomplished by setting the wrench to install at a pretension well above the required pretension, yet not high enough to exceed the turn-of-nut table. This same wrench setting could be tested on another length of bolt of the same diameter and grade, and then another. Perhaps one wrench setting could be used throughout the day for all bolts of the same diameter and grade being installed, or perhaps only a few wrenches would be needed instead of several.

Snugging the joint can be done with either the calibrated wrench (actually in the uncalibrated condition, releasing the trigger when snug is achieved), with an impact wrench, or with a hand wrench for lighter framing. After snugging, the joint should be inspected to verify snug. After the wrenches are calibrated, pretensioning can begin. The wrench operator should tighten the bolts using a systematic pattern, observing the chuck rotation as tightening proceeds. If the rotation of the nut exceeds the turn-of-nut table, the wrench calibration should be rechecked.

8.3.3 Direct tension indicator installation method

The *direct tension indicator,* or *dti*, is a load-cell device used to establish that the required pretension has been provided in the assembly. The manufacturing and testing of the dti itself is governed by ASTM F959. The effectiveness of the dti, however, is also dependent upon the techniques used in installing the fastener.

The dti has protrusions (bumps) formed into the device that will be partially compressed when the bolt is pretensioned. The gap remaining between the dti face and the fastener element against which it is placed should not close below a specified gap until after the fastener has reached the required fastener pretension. By use of a feeler gage or the naked eye to verify that the gaps have been suitably reduced, one can verify that the bolt has been pretensioned.

The preferred position of the dti is with the bumps facing outward directly underneath the bolt head. During installation, the bolt head must be held from turning to prevent abrasion of the bumps, thereby rendering the dti measurement invalid. For building applications, this condition would call for a 0.015-in (0.38-mm) feeler gage to be refused entry in half or more of the gaps of the dti. For bridges, a 0.005-in (0.13-mm) feeler gage is used. If the dti is coated rather than uncoated, a 0.005-in (0.13-mm) feeler gage is used for all purposes.

If the bolt head will be turned, a hardened F436 washer must be placed between the bolt head and dti. If the dti is placed at the nut end of the assembly, an F436 washer must be used between the dti and nut, whether or not the nut is allowed to turn. In each of these cases, a 0.005-in (0.13-mm) feeler gage is used to check the gaps.

The dti bumps must face outward away from the steel to keep the dti from cupping outward, opening the gaps larger and voiding the measurement technique. Standard F436 washers are needed behind the dti when the dti is used over an outer ply containing an oversized or slotted hole. This prevents the dti from cupping into the hole and voiding the gap-measurement technique.

The joint is first snugged using a systematic technique, then inspected. The installer should check that at least half the gaps do not refuse the feeler gage. This is done to ensure that the bolt did not reach its required pretension during snugging, then subsequently loosen when adjacent bolts were snugged. Since the dti is inelastic, it will not rebound to increase the gap when the preload is released. Therefore, a bolt "overtightened" during snugging, then subsequently loosened, will still appear to be properly pretensioned by having an adequate number of refusals for the postinstallation check. After the entire joint has been verified as snug, the installation crew can then proceed to pretension each bolt until at least half the dti bumps refuse entry of the feeler gage.

8.3.4 Twist-off-type tension-control bolt installation method

The *twist-off-type tension-control bolt* incorporates a specially designed bolt that has a spline at the end that is used by the installation wrench. The spline is designed to shear off because of the torque generated by the wrench. This torque is the result of wrench efforts to turn the nut in the clockwise (tightening) direction, resisted by counteracting efforts to turn the bolt shank in the counterclockwise direction. When the twist-off bolt's head grabs the steel, the bolt shank itself will not turn, and all rotation takes place with the nut. The spline has been designed so that it will not shear off until the bolt is above the required pretension. When the spline shears off, the wrench no longer functions.

The twist-off bolt is completely dependent upon the torque-tension relationship, which can vary greatly depending upon the quality and type of lubrication, if any, of the assembly. The manufacturers of twist-off bolts generally use a very consistent and durable lubricant that resists water, mild solvents, and rust for some time, but performance may be affected by heat, cold, and moisture. Some manufacturers rely upon the natural lubrication provided from the quenching medium.

Because of the interdependence of the bolt, nut, and washer upon the torque used for installation, the twist-off bolt unit is preassembled by the manufacturer. Substitutions of other nuts or washers may adversely affect performance and cause bolt pretensions to be too high or too low, or cause bolt failure.

The joint is first snugged using a systematic method, as with all installation procedures previously discussed. Care must be used to make sure that the spline is not twisted off during the snugging operation. Any bolts that twist off during snugging must be replaced. In some cases, deep sockets are used with conventional impact wrenches to snug the joints, therefore protecting the splines. Upon completion of snugging, the snug condition is verified. Upon acceptance of the snugged joint, the installation crew proceeds to pretension each twist-off bolt with the installation wrench until the spline shears off. A systematic pattern should be used for this step.

8.3.5 Lock pin and collar installation method

A form of alternate design fastener is the lock pin and collar fastener. With this type of fastener, a pin with a set of concentric locking grooves and a break-neck is pretensioned with a hydraulic tool. As the tool pulls on the pin, the unit also swages a locking collar onto the locking grooves to retain the pin's pretension. At a point above the required pretension for the fastener, the break-neck portion of the pin fractures in tension, stopping the pretensioning process. The pin maintains the required residual pretension because of the locking collar.

8.3.6 Bolt inspection procedures

Bolt inspection is a multistep operation that begins before the bolt installation starts. See Sec. 8.2 for preinstallation inspection steps (1) through (7). The following additional steps should be included in a bolt inspection program:

1. Visually check after snugging to verify that the snug condition has been achieved. For dti's, also verify that the installers check for an adequate number of remaining gaps in their dti's. For twist-off bolts, check that the splines are intact.
2. Observe the installation crew for proper technique. Observation of the installation crew does not mean that the installation of each individual bolt is observed, but that the crew is observed to verify their knowledge and understanding of procedures, and follow the proper pretensioning procedures on a consistent basis.

8.3.7 Arbitration of disputes

Arbitration is applicable only when there is a dispute, per Section 10 of the RCSC specification. The methodology given is not intended as an inspection method, but as a way to resolve claims that the crew may not have followed the proper techniques for a particular joint. Arbitration is not to be used as a substitute for inspection per RCSC specification Section 9—the visual observation of the preinstallation testing, checking for snug, and observation of the installation technique of the crew.

For bridge work, torque inspection testing is still required for 10% of the bolts in each connection, minimum two per connection, for bolts installed using turn-of-nut or calibrated-wrench methods. The torque-testing procedures of AASHTO are similar to the RCSC specification Section 10, except that only three bolts are used to determine the inspection torque, not five. For dti, twist-off-type tension-control bolt, and lock pin and collar installations, no torque testing is required.

8.4 Bolt Inspection Issues

8.4.1 Hole punching and drilling

The AISC specification addresses the use of punching, drilling, and thermal cutting for making bolt holes. The size of hole used for a particular size bolt may vary with the type of joint and hole selected by the engineer. Table 8.2 states the given hole sizes for each diameter of bolt, as required by AISC Table J3.3.

Oversized holes may be used only in slip-critical joints. Slotted holes may be used in snug-tightened and pretensioned joints only when the load is transverse to the direction of the slot. Otherwise, they may be used only in slip-critical joints.

TABLE 8.2 Nominal Bolt Hole Dimensions

Bolt Diameter	Hole dimensions, in			
	Standard	Oversized	Short slot	Long slot
$^1/_2$	$^9/_{16}$	$^5/_8$	$^9/_{16} \times {}^{11}/_{16}$	$^9/_{16} \times 1^1/_4$
$^5/_8$	$^{11}/_{16}$	$^{13}/_{16}$	$^{11}/_{16} \times {}^7/_8$	$^{11}/_{16} \times 1^9/_{16}$
$^3/_4$	$^{13}/_{16}$	$^{15}/_{16}$	$^{13}/_{16} \times 1$	$^{13}/_{16} \times 1^7/_8$
$^7/_8$	$^{15}/_{16}$	$1^1/_{16}$	$^{15}/_{16} \times 1^1/_8$	$^{15}/_{16} \times 2^3/_{16}$
1	$1^1/_{16}$	$1^1/_4$	$1^1/_{16} \times 1^5/_{16}$	$1^1/_{16} \times 2^1/_2$
$\geq 1^1/_8$	$d + {}^1/_{16}$	$d + {}^5/_{16}$	$(d + {}^1/_{16}) \times (d + {}^3/_8)$	$(d + {}^1/_{16}) \times (2.5d)$

Bolt Diameter	Hole dimensions, mm			
	Standard	Oversized	Short slot	Long slot
M16	18	20	18 × 22	18 × 40
M20	22	24	22 × 26	22 × 50
M22	24	28	24 × 30	24 × 55
M24	27	30	27 × 32	27 × 60
M27	30	35	30 × 37	30 × 67
M30	33	38	33 × 40	33 × 75
≥ M36	$d + 3$	$d + 8$	$(d + 3) \times (d + 10)$	$(d + 3) \times (2.5d)$

For bridges, AASHTO limits the punching thickness to $^3/_4$ in (19 mm) for structural steel, $^5/_8$ in (16 mm) for high-strength steel (50 ksi and more), and $^1/_2$ in (13 mm) for quenched and tempered steel. If the thickness of the steel exceeds these limits, the holes must be drilled or subpunched or subdrilled and then reamed to the proper diameter. AASHTO also requires that holes penetrating through five or more layers of steel either be subdrilled and reamed to the proper diameter or drilled full size while preassembled.

The size of the completed hole may exceed the nominal diameter of the hole by a maximum of $^1/_{32}$ in (1 mm). If the hole size is larger, then it must be considered oversized, which may change the design assumption, the design strength for the bolt, and the bearing strength of the steel.

8.4.2 Bolt storage and control

Bolts, nuts, and washers are typically purchased as commodity items and are placed into inventory. Because the shop bolt list is not completed until the shop detail drawings are done, and the field bolt list is not done until the erection plans and shop details are done, bolts are ordered in advance using estimates of quantities and lengths.

Bolts, nuts, and washers should be maintained in protected storage with the manufacturer's certification available. Because the RCSC specification requires preinstallation testing for fastener assemblies by production lot; bolts and nuts should not be mixed with those of other production lots. If torque-control methods are used for installation, inventory control by production lot is especially important.

Only a few fastener manufacturers place their lot number on the fastener itself. All others place their lot identification on the keg or box only. Once removed from the container, lot identification can be maintained only through established shop or field-control procedures.

8.4.3 Lubrication

All black (plain) bolts must be adequately lubricated when installed. Rather than applying a lubricant, most manufacturers supply black bolts, nuts, and washers with the residual water-soluble oil from the quenching operation or manufacturing operation. If the fasteners are exposed to rain, snow, dew, condensation, or other moisture conditions, this oil may be washed off. This oil may also evaporate after a period of time when left in open containers.

It is a specification requirement that black fasteners be oily to the touch prior to being installed. When compared to oily fasteners, bolts that have lost their lubrication may require as much as twice the torque to install them, requiring more time and more powerful tools. In addition, the bolt's ductility is reduced because of the higher torque needed to tighten poorly lubricated fasteners.

Should bolts, nuts, or washers show rust, the rust must be cleaned from the surface of the fastener component and then the component must be lubricated or relubricated. Dirt, sand, grit, and other foreign material must be cleaned off the fastener prior to installation, with lubrication added when necessary.

If a bolt, nut, or washer has lost its existing lubrication, it is required that the component be lubricated prior to installation. The type of lubrication to be used is not specified, but typically an oil-based product, stick wax, bee's wax, liquid wax, or spray lubricant can be used.

The most effective lubrication is placed on the threads of the nut and on the face of the nut that will be turned against the steel or washer. Approximately 60% of the torque used to tighten a bolt is used to overcome the friction between the nut and either the washer or the steel. Another 30% of the torque is used to overcome the friction between bolt threads and nut threads. Only about 10% of the installation torque is the energy used to actually induce bolt pretension.

In some cases for black bolts, added lubrication or relubrication mandates the retesting of fasteners in a bolt-calibration device prior to installation in the structure. This verifies the effectiveness of the new lubrication. Highly efficient lubricants can actually increase the risk of thread stripping, so this condition is also checked.

If the calibrated-wrench method is used for installation, any new lubrication mandates the recalibration of the installation wrenches. The efficiency of lubrication rarely negatively affects the performance of bolts using the turn-of-nut or direct tension indicator methods of installation.

The RCSC specification severely limits lubrication of the twist-off bolt assembly by anyone other than the manufacturer. Many twist-off bolts use a special lubricant that is not as oily as common structural bolts. Contacting the manufacturer or supplier of the twist-off bolt system is required prior to lubrication or relubrication. These fasteners are particularly sensitive to inadequate lubrication and overlubrication, and loose bolts or broken bolts may result.

Galvanized fasteners, either hot-dipped galvanized or mechanically galvanized fasteners, are lubricated in a manner different than black bolts. They are not oily. The nut is the only lubricated component of the assembly. The nut must receive from the manufacturer a coating that is clean and dry to the touch. Usually a wax-based product is used, but the wax's presence may not always be determined by touch. Often, a dye is added to the lubricant to verify that the nuts have indeed been lubricated. Sometimes, a UV solution is used in the lubricant to make the nut "glow" under an ultraviolet light. If the presence of a lubricant is uncertain, torque testing in a bolt tension–calibration device will provide indication of the lubrication's presence. For bridge work, if the torque required to tighten the assembly is less than the maximum torque permitted in the AASHTO rotational-capacity test, then the nut has been adequately lubricated.

If new lubrication is required, a wax-based or similar lubricant works well. Apply the lubricant to the threads of the nut and to the inside face of the nut. It is not necessary to lubricate the bolt or washer when this is done. After lubrication, test the assembly in a bolt tension–calibration device for torque performance and resistance to stripping.

8.4.4 Bolt stickout

Stickout is the amount of bolt thread sticking out beyond the face of the nut after tightening. The RCSC specification requirement is that the end of the bolt be at least flush with the face of the nut. The bolt end cannot be below the face of the nut after tightening is completed.

There is no maximum stickout by specification, but excessive stickout indicates a risk that the nut has actually met the thread runout.

If this has occurred, pretensioning is questionable for the calibrated wrench and twist-off installation methods because the nut would cease rotation and the torque would become very high, although the bolt would remain loose. For the turn-of-the-nut method, the required turns could not be applied. For the dti method, the dti gap requirements could not be achieved.

For pretensioned bolts, a second danger of maximum stickout is that the risk of thread stripping is increased. The bolt threads will neck down in a very short region when the bolt is pretensioned, reducing the thread contact between bolt and nut.

Excessive stickout measurement is determined by the actual bolt and nut combination, and can be checked visually using an untightened bolt with the nut run up to the bolt thread runout. Generally, six threads of stickout can be permitted for $^1/_2$-, $^5/_8$-, $^3/_4$-, and $1^1/_8$- in bolts. For $^7/_8$-, 1-, $1^1/_4$-, and $1^3/_8$-in bolts, five threads of stickout can be permitted; and for $1^1/_2$-in bolts, four threads can be permitted. Stickout exceeding these values should be checked with the comparison set, and may be found acceptable.

Bolt ductility is highest when the nut is flush with the end of the bolt because of the maximum number of threads available for stretching. With maximum stickout, the bolt's ductility is reduced because the stretch is limited to the very short length of thread in the grip.

A traditional "rule of thumb" had been to require two threads of stickout for high-strength bolts. This was a guideline developed for applications where the thread-excluded condition was specified. It is neither a valid indicator that the thread-excluded condition has been achieved, nor is it required by specification; therefore, it should not be part of an inspection requirement.

8.4.5 Washers

The RCSC specification Section 6 provides the following situations where F436 hardened steel washers and other special washers are required. Washers are suggested, even for cases when not required, to ease installation and provide better consistency for installation and inspection.

1. Washers are not required for snug-tightened joints, and pretensioned joints when using the turn-of-nut method or the direct tension indicator method, if only standard holes are present in the outer plies.
2. For snug-tightened joints, or pretensioned joints using the turn-of-nut method or the direct tension indicator method, with slotted holes present in an outer steel ply, an F436 washer or common plate washer is required over the slot.

3. If the slope of the face of the connected part exceeds 1:20, or 3°, relative to the bolt or nut face, a hardened beveled washer must be used between the fastener and the steel to compensate for the slope.

The following provisions apply only to pretensioned and slip-critical joints:

1. If the calibrated-wrench method is used, an F436 washer must be used directly under the turned element.
2. If twist-off-type tension-control bolts are used, the supplier's washer must be used directly under the nut.
3. If A490 bolts are used with A36 steel (or another type of steel, below 40-ksi yield strength) as the connection material, an F436 washer must be provided under both the bolt head and the nut.
4. If oversized or short-slotted holes are used in an outer steel ply, and the bolts are A325 of any diameter or A490 of 1-in diameter or less, an F436 washer must be placed over the hole or slot.
5. If oversized or short-slotted holes are used in an outer steel ply, and the bolts are A490 over 1-in diameter, an F436 washer of minimum $^5/_{16}$-in thickness must be placed over the oversized hole or slot. Multiple standard thickness F436 washers cannot be substituted for the thicker single washer.
6. If a long-slotted hole is used in an outer steel ply, and the bolts are A325 of any diameter or A490 of 1-in diameter or less, a plate washer or continuous bar of minimum $^5/_{16}$-in thickness with standard holes must be used to cover the slot. The bar or plate material must be of structural grade but need not be hardened.
7. If a long-slotted hole is used in an outer steel ply, and the bolts are A490 of over 1-in diameter, a $^3/_8$–in-thick bar or plate washer with standard holes, with an F436 washer placed between bar or plate and fastener, must be used. The bar or plate material must be of structural grade but need not be hardened.
8. If a twist-off bolt having a round head with a diameter at least equal to that required by ASTM F1852 or F2280 is used, and the preceding provisions call for a standard thickness F436 washer, no washer is required under the bolt head.

8.4.6 Systematic tightening

The specifications require that joints be snugged and tightened in a systematic manner. A pattern should be chosen for tightening the bolts so that the joint is drawn solidly together. The pattern should also be used so that bolts are not inadvertently missed during snugging or pretensioning.

The joint should be snugged first, starting at the most rigid part of the joint. In a joint with a single or double row of bolts, this would be where the steel is already in contact, working toward the end where the steel may not be in contact. If there is solid contact between the steel at all locations, the direction of tightening does not matter. In a bolt pattern with several rows, such as a large splice plate, the bolts in the center of the joint should be snugged first; then proceed to work toward the free edges of the plate.

After the joint has been completely snugged, pretensioning of the bolts should follow the same systematic pattern used for snugging.

8.4.7 Reuse of bolts previously tightened

Occasionally, it may be necessary to remove a previously pretensioned bolt and later reinstall it. The specification permits reuse of black A325 bolts only with the engineer's permission. Galvanized bolts and A490 bolts cannot be reused in any case.

Bolts that have been installed to the snug condition, then subsequently loosen when adjacent bolts are snugged, are not considered as reused bolts. Similarly, bolts that are touched up in the pretensioning process are not considered reused. To be considered as reuse, the bolt must have been pretensioned, then loosened.

To check previously pretensioned black A325 bolts to see whether they can be reused, run the nut up the entire length of the bolt threads by hand. If this is possible, the bolt may be reused. Bolts that have yielded from tightening will stretch in the first few threads (nearest the bolt head), preventing the nut from progressing further up the threads. These bolts should not be reused.

Because of the overtapping of the nut threads for galvanized fasteners, this check is not valid for galvanized bolts. A490 bolts do not have the same ductility as A325 bolts, therefore A490 bolts may not be reused.

8.5 Inspection Prior to Welding

The responsibilities and levels of welding inspection must be established in the contract documents. Neither the American Welding Society (AWS) or AISC, nor the model-building codes, provide a complete and comprehensive listing of all welding inspection duties. Inspection duties may be assigned to the contractor (fabrication/erection inspection) or to an inspector who reports to the owner or engineer (verification inspection). Under the special inspection requirements of the *International Building Code,* certain welding inspection must take place by an inspector responsible to the owner or engineer.

The welding inspection provisions of AWS do not assign specific inspection tasks to either the contractor or verification inspector, with a few exceptions. Rather, inspection as stated means primarily the contractor's (fabricator's or erector's) inspector. Verification inspection is not mandated. Any forms of nondestructive testing must be specified in the contract documents, as AWS requires only visual inspection.

Both general and specific welding inspection requirements are provided in AWS D1.1. Clause 6 on "Inspection" covers procedural matters and acceptance criteria. Some workmanship requirements are found in Clause 5 on "Fabrication." Various inspector checklists have been compiled and published in numerous sources.

Welding inspection is a start-to-finish task. Inspection can be broken into three timing categories: before welding, during welding, and after welding.

8.5.1 Welding processes

Four welding processes predominate in structural steel fabrication and erection. These are

1. Shielded metal arc welding (SMAW)
2. Flux-cored arc welding (FCAW)
3. Gas metal arc welding (GMAW)
4. Submerged arc welding (SAW)

In addition, three other processes may be used from time to time. Electroslag welding (ESW) and electrogas welding (EGW) may be used for large, thick weldments in some applications. Gas tungsten arc welding (GTAW), also commonly called tungsten inert gas welding (TIG), may be used for small welds, thin sheet steel materials, and for joining specialty steels.

The first four processes (SMAW, FCAW, GMAW, and SAW) are considered prequalified welding processes under AWS D1.1 Clause 3.2.1. However, the short-circuiting transfer mode of GMAW, abbreviated GMAW-S, is not prequalified. The benefit of prequalified welding processes is that no preproduction testing of the welding procedure specification (WPS) is required for a particular joint, provided the joint design, WPS, steel, and filler metal fall within the limits of the AWS D1.1 Clause 3. Nonprequalified procedures require testing of the WPS, with a written welding procedure qualification record (PQR) document for the test.

Shielded metal arc welding The shielded metal arc welding (SMAW) is a common welding process used for a variety of applications. The electrode is a fixed-length rod of a given steel and diameter, covered by a

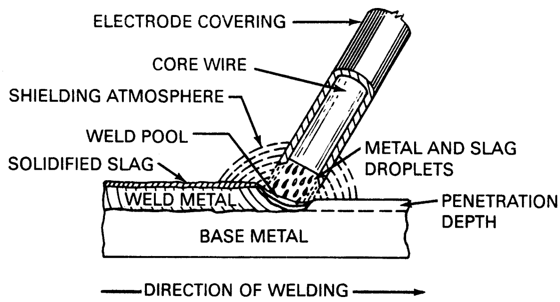

Figure 8.1 Shielded metal arc welding. (*Adapted from American Welding Society Welding Handbook, vol. 2.*)

coating that provides a shielding and fluxing agent, and sometimes supplies alloy content (see Fig. 8.1). The electrical current passes through the electrode to the steel, or from the steel to the electrode, depending upon polarity, forming an arc between the electrode and the steel. The heat of the arc melts a portion of the steel and the end of the electrode. The electrode steel, and alloying material is transferred to the weld puddle by the electrical forces generated. The electrode material and steel are mixed together by the arc action, then solidify to form the weld metal. The fluxing agent supplied by the electrode coating, as well as impurities generated by the welding itself, solidify in the form of slag on the top surface of the weld.

SMAW electrodes are categorized as low hydrogen and non–low hydrogen. Low hydrogen electrodes have coatings designed to provide a weld and heat-affected zone with minimal diffusible hydrogen. Hydrogen is a contributor to underbead cracking. Low hydrogen electrodes are identified as EXXX5, EXXX6, and EXXX8. Low hydrogen electrodes also require special care in storage and handling, possibly including baking and drying, to retain their low hydrogen characteristics.

Electrodes can be from AWS A5.1 "Specification for Carbon Steel Electrodes for Shielded Metal Arc Welding" or AWS A5.5 "Specification for Low-Alloy Electrodes for Shielded Metal Arc Welding." AWS A5.5 electrodes are optional for steels of groups I and II as listed in AWS D1.1 Table 3.1. They are required for AWS group III and IV steels and the high-strength quenched and tempered steels listed in AWS D1.1 Table 4.9, unless WPS qualification testing is performed. The nomenclature used to identify SMAW electrodes is shown in Table 8.3.

Flux-cored arc welding Flux-cored arc welding (FCAW) is another popular prequalified welding process. The process uses a tubular wire containing fluxing and alloying agents inside the wire. The wire is fed

TABLE 8.3 Electrode Classification System for Sheilded Metal Arc Welding

EXXXX-X	
E	Electrode
XX	Minimum tensile strength of undiluted weld metal, ksi, in the as-welded condition
X	Permitted position 1. All 2. Horizontal (fillets only) and flat 4. Flat, horizontal, overhead, and vertical down
X	Coating and operating characteristics 5, 6, and 8—low hydrogen
-X	Major alloying elements (A5.5 electrodes)

into the weld area using a welding "gun." The heat of the arc vaporizes the fluxing agents in the core, releases the alloying elements, melts the electrode wire to enable transfer by the arc, and melts the steel being welded in the area of the arc (see Fig. 8.2).

When used without gas shielding, so-called self-shielded, the process is designated FCAW-S. FCAW with gas shielding is designated FCAW-G. Because of the gas shielding, welding must not be done in wind velocities greater than 5 mi/h (8 km/h) under the provisions of AWS D1.1 Clause Section 5.12.1.

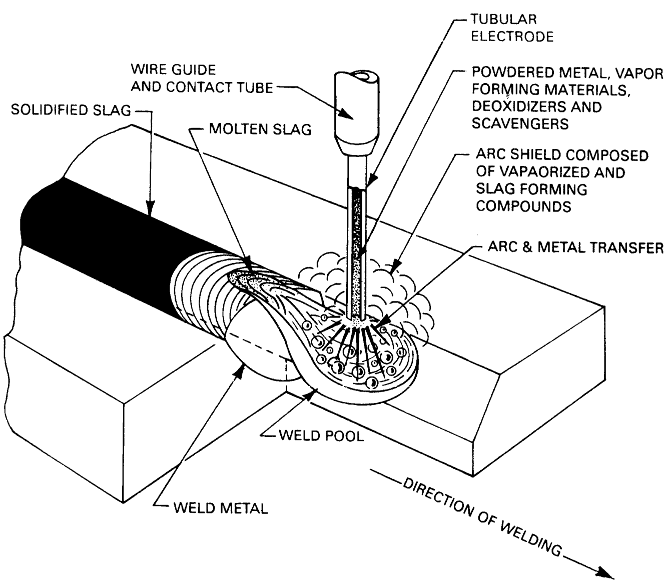

Figure 8.2 Flux-cored arc welding. (*Adapted from American Welding Society Welding Handbook, vol. 2.*)

TABLE 8.4 Electrode Classification System for Flux Cored Arc Welding for A5.29 Low-Alloy Electrodes

EXXTX-X	
E	Electrode
X	Minimum tensile strength of deposited weld metal, 10 ksi, in the as-welded condition
X	Permitted position 0. Flat and horizontal 1. All positions
T	Tubular wire
X	Electrode classification
-X	Alloy information

Flux-cored electrodes for structural steel may be of two types, AWS A5.20 "Specification for Carbon Steel Electrodes for Flux Cored Arc Welding" or AWS A5.29 "Specification for Low-Alloy Steel Electrodes for Flux Cored Arc Welding." Flux-cored electrodes are designated as shown in Table 8.4.

FCAW is known for higher productivity rates because the wire is continuously fed from a spool or drum, compared to the fixed-length electrodes of SMAW, and because of higher deposition rates.

Gas metal arc welding Gas metal arc welding (GMAW) is similar to FCAW in that the electrode wire is continuous and welding is performed using a welding "gun." The GMAW wire is a solid wire, rather than tubular one, like FCAW. In some applications, a composite wire, also called metal-cored wire, may be used. The shielding of the weld region is supplied by an external shielding gas (see Fig. 8.3). Gas metal arc welding is also commonly referred to as MIG (metal inert gas) welding, or as MAG (metal active gas) welding, depending upon the type of shielding gas used.

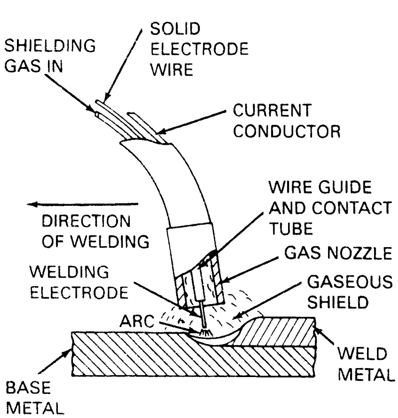

Figure 8.3 Gas-metal arc welding. (*Adapted from American Welding Society Welding Handbook, vol. 2.*)

TABLE 8.5 Electrode Classification System for Gas Metal Arc Weld For A5.28 Low-Alloy Electrodes

ERXXS-X	
ER	Electrode rod (if ER, may also be used as GTAW filler rod; if E, may not be used for GTAW)
XX	Minimum tensile strength of deposited weld metal, ksi, in the as-welded condition
S	Solid wire (C designates metal-cored wire)
-X	Alloy information (A5.28)

GMAW electrodes can be of two types: AWS A5.18 "Specification for Carbon Steel Electrodes and Rods for Gas Shielded Arc Welding" or AWS A5.28 "Specification for Low Alloy Steel Electrodes and Rods for Gas Shielded Arc Welding." AWS A5.28 electrode wires must be used for AWS D1.1 Table 3.1 group III and IV steels and AWS D1.1 Table 4.9 high-strength quench and tempered steels, unless otherwise qualified by test. Electrodes for gas metal arc welding are identified as shown in Table 8.5.

Common shielding gases used for structural steel include a mixture of argon and carbon dioxide, and straight carbon dioxide. Similar to FCAW-G, GMAW welding cannot be performed in wind velocities greater than 5 mi/h (8 km/h); therefore, it is rarely used in field applications. Shielding gases are addressed in AWS A5.32 "Specification for Welding Shielding Gases."

GMAW has two advantages for welding structural steel; there is minimal slag to be removed, and there are reduced welding fumes to be exhausted from the work space.

Submerged arc welding The submerged arc welding (SAW) employs a wire-fed welding gun and a deposit of granular flux. The arc is buried beneath the blanket of granular flux, shielding the weld region from atmospheric impurities, as well as providing fluxing and alloying elements (see Fig. 8.4). As an automatic or semiautomatic process, it is commonly the fastest prequalified welding process once setup is complete. Because the weld puddle is buried beneath the flux, not visible to the welder, establishing proper welding parameters and accurately tracking the root of the weld is critical to the satisfactory completion of the weld.

Submerged arc welding may be performed with multiple welding electrodes, set either in parallel, with both electrodes controlled by the same feeder and power supply, or in tandem, with separate feeders and controls.

The proper combination of electrode wire and flux is important for quality welding. For this reason, the electrode and flux combination is specified within one document. AWS A5.17 covers "Specification for Carbon Steel Electrodes and Fluxes for Submerged Arc Welding," and

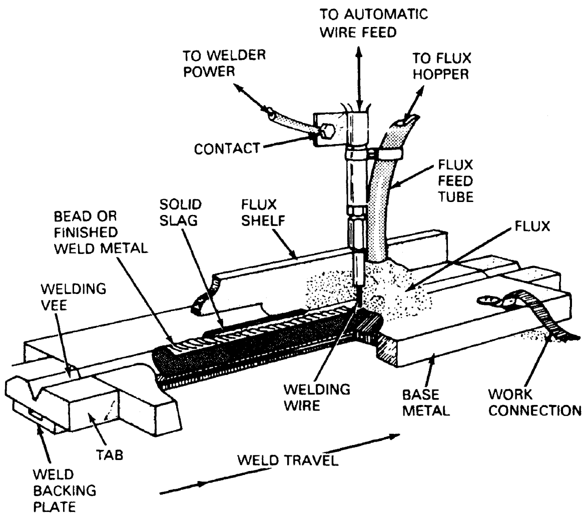

Figure 8.4 Submerged arc welding. (*Adapted from American Welding Society Welding Handbook, Vol. 2.*)

AWS A5.23 covers "Specification for Low-Alloy Electrodes and Fluxes for Submerged Arc Welding." The nomenclature system used for SAW electrodes and fluxes under AWS A5.17 is shown in Table 8.6. AWS A5.23 specifications use a more complex nomenclature system to provide additional information on the system.

TABLE 8.6 Flux and Electrode Classification System for Submerged Arc Welding

FXXX-EXXX	
F	Flux
X	Strength level intended for flux, 10 ksi units
X	A or P, A for weld deposit strength determined in the as-welded condition, P for strength determined in postweld heat-treated condition
X	Impact properties temperature Z. No impact properties 0. 20 ft-lb at 0°F (27 J at –18°C) 2. 20 ft-lb at –20°F (27 J at –29°C) 4. 20 ft-lb at –40°F (27 J at –40°C) 6. 20 ft-lb at –60°F (27 J at –51°C)
-E	Electrode (EC for composite electrode)
X	Manganese content L. Low manganese (0.6% max.) M. Medium manganese (1.40% max.) H. High manganese (2.25% max.)
X	Nominal carbon content, in 1/1000% Ex: 12=0.12%
X	Killed steel (if applicable)

8.5.2 Welding procedures

The use of a written welding procedure specification (WPS) is mandated by AWS D1.1 Clause 5.5. The WPS may be either prequalified or qualified by test. A prequalified WPS must fall within the limits prescribed in Clause 3 of AWS D1.1 (notably Table 3.7), must use a prequalified welding process, and must be on a steel and in a joint deemed prequalified in Clause 3. All other WPSs must be qualified by test using the procedures set forth in Clause 4 of AWS D1.1. Annex. N of AWS D1.1 provides sample WPS forms.

Welding procedure specifications may be furnished by electrode suppliers, welding equipment suppliers, technical organizations, or consultants, but are most frequently developed by the contractor. WPSs are specific to the following parameters:

Welding process

Base metal (steel classification, strength, type)

Base metal thickness (range)

Joint type (butt, tee, corner)

Weld type (groove, fillet, plug)

Joint design details (root opening, groove angle, use of backing)

Use of backgouging

Position (flat, horizontal, vertical, overhead, tubular application)

Using these parameters, the following items are established:

Electrode classification

Electrode diameter

Flux classification

Shielding gas

Shielding gas flow rate

Voltage

Amperage or wire feed speed

Polarity

Electrical stick-out

Travel speed

Number and position of passes

Technique

Preheat, interpass, and postheat requirements

Cleaning requirements

The inspector should check for and review the WPSs to be used for the project. The WPS is specific to the material, welding process, position, type of joint, and configuration of joint. He should also verify that a proper WPS is available for all welds to be completed, and that the welding personnel follow the WPS as prescribed.

8.5.3 Welding personnel

Welders, tack welders, and welding operators must be qualified by the contractor responsible for the welding prior to the welding being performed, as required by AWS D1.1 Clause 4.1.2.

Welders are individuals who manipulate the welding electrode, wire and/or filler metal by hand to make the weld, so-called manual or semiautomatic welding. A welding operator sets up and adjusts equipment to perform automatic welding, done without manual manipulation of the elctrode. A tack welder is a fitter who makes small welds as necessary to hold parts together until final welding by a welder or welding operator.

The fabricator or erector responsible for welding must have each welder, tack welder, and welding operator tested using the methods of AWS D1.1 Clause 4, Part C, to prove their capability to make adequate quality welds. The testing may be performed by the contractor (fabricator or erector) or by an independent testing laboratory. These individuals are tested and categorized by

Welding process

Welding position

Weld type

Base metal thickness range

Electrode classification group (if SMAW)

Welder performance qualification records must be made available for the inspector's review prior to the start of welding. If a previous employer's testing results are to be used, the engineer must approve the current employer's reliance upon these previous tests.

Welders who perform and pass such testing at an independent testing laboratory accredited by the American Welding Society can have their test records placed on file with the AWS and receive the designation of "AWS Certified Welder."

A welder's or welding operator's qualification for a given employer remains in effect indefinitely, as long as that individual continues welding in that given process, although not necessarily with the tested electrode classification (for SMAW) or in the tested position. If the welder fails to use that process for a period exceeding 6 months, the welder must complete and pass a welding test. If the welder's quality becomes subject to question, the welder's qualification may be revoked, forcing a retest. Tack welders' qualifications remain in effect perpetually, unless there is specific reason to question the tack welder's abilities. Although welder qualification is the responsibility of the contractor, under the provisions of AWS D1.1 Clause 6.4.2, the inspector may also force requalification testing if the welder's quality is poor.

8.5.4 Base metal quality

The inspector should verify that the quality of the base metal is suitable for welding. The steel to be welded must be clean, smooth, and without surface discontinuities such as tears, cracks, fins, and seams. Such surface discontinuities could propagate into the weld after welding. The surface should also be free of excessive rust, mill scale, slag, moisture, grease, oil, and any other material that could cause welding problems. Some materials may be permitted, such as thin mill scale (mill scale that withstands a vigorous wire brushing), thin rust-inhibitive coatings, and antispatter compounds made specifically for weld-through applications. AWS D1.1 Clause 5.15 provides additional information and exceptions to these provisions.

8.5.5 Joint preparation and fit-up

Fillet weld fit-up tolerances are given in AWS D1.1 Clause 5.22.1. Gaps between parts of $^{1}/_{16}$ in (1.6 mm) or less are permitted without correction. If the gap exceeds $^{1}/_{16}$ in (1.6 mm) but does not exceed $^{3}/_{16}$ in (5 mm), then the leg dimensions of the fillet weld are to be increased to compensate for the gap between the parts. Gaps over $^{3}/_{16}$ in (5 mm) are permitted only in materials more than 3 in (76 mm) in thickness. In these cases, the use of a backing material is required as well as compensation in the weld leg dimensions. Such provisions are not to be used for gaps of more than $^{5}/_{16}$ in (8 mm). Similar provisions are used for partial joint-penetration groove welds when the welds are parallel to the length of the member.

When groove welds are used, tolerances to the root opening, groove angle, and root face apply. The specific tolerances depend upon the type of groove weld, the presence of backing, and the use of backgouging. AWS D1.1 Clause 5.22.4 and AWS D1.1 Figure 5.3 provide these values. Groove tolerances are also provided in the prequalified groove weld details in AWS D1.1 Figures 3.3 and 3.4. For tubular joints, AWS D1.1 Clause 5.22.4.2 governs.

Part alignment for butt joints can be critical, depending upon application. AWS D1.1 Clause 5.22.3 requires alignment within 10% of the part thickness, not to exceed $^1/_8$ in (3 mm), when the parts are restrained from bending from such misalignment. No provisions are given for cases where such restraint does not exist. For girth welds in tubular joints, the alignment tolerances are provided in AWS D1.1 Clause 5.22.4.

8.5.6 Welding equipment

In order to properly follow the parameters of a welding procedure specification, the equipment used for welding must be in good condition and of adequate capacity to make the specified welds at the duty cycle employed. The inspector should, if necessary, use testing equipment to verify that the equipment settings and the welding machine output at the point of welding satisfy the WPS.

8.5.7 Welding consumables

Welding electrodes, fluxes, and shielding gases should be checked to be in conformance with AWS D1.1 Clause 5.3. Low hydrogen SMAW electrodes require special controls, including requirements for storage temperatures and exposure time limits. Fluxes for SAW require dry, contamination-free storage, with the removal of the top 1 in (25 mm) of material from previously opened bags at the beginning of each day, prior to use. Drying of flux from damaged bags may be required.

Shielding gases must be of welding grade and have a dew point of −40°F (−40°C) or lower. The gas manufacturer's certification of dew point may be required by the engineer.

Welding materials such as electrodes and fluxes should have manufacturers' certificates of compliance that they meet the applicable American Welding Society standard. These certificates of compliance may be requested by the engineer.

8.5.8 Welding conditions

For good welding, the welder and the operating equipment must have conditions suitable for welding, within given in AWS D1.1 Clause 5.12. The temperature in the area immediately surrounding the welding must be above 0°F (−18°C). The ambient temperature in the general vicinity can be lower, but heating must be provided to raise the temperature immediately around the weld to at least this temperature. The surfaces to be welded must not be wet or exposed to moisture. High winds must be avoided. For GMAW, GTAW, EGW, and FCAW-G, the wind velocity must not exceed 5 mi/h (8 km/h), requiring protective enclosures in most field applications. No maximum wind velocity is specified for welding processes requiring no shielding gases, but a practical limit is around 25 mi/h (40 km/h).

8.5.9 Preheat

Preheating of the steel is necessary for thick steels, certain high-strength steels, and steels when their temperature is below 32°F (0°C). The preheating requirements should appear in the welding procedure specification. Prequalified minimum preheat requirements are provided in AWS D1.1 Table 3.2. In this table, the required preheat is given for a specified steel specification group, welding process, and thickness range of part. When the temperature of the steel is below 32°F (0°C), the steel must be heated to at least 70°F (21°C). The thicker the steel, the higher the required preheat temperature. Higher-strength steels also require higher preheats. Certain high-strength steels have preheats limited to a maximum of 400°F (205°C). Preheat requirements may also be modified using the provisions of AWS D1.1 Annex. I, which evaluates the requirements based upon the diffusible hydrogen of the filler metal, joint restraint, and the weldability (carbon equivalency) of the steel.

8.6 Inspection During and After Welding

After checking the welding personnel qualifications, WPS, welding consumables, steel materials, welding conditions, equipment, joint fit-up, and preheat, the welding inspection performed during welding is limited to verifying that the welding procedures are properly followed. This includes the maintenance of interpass temperature during welding, usually the same temperature as required for preheat. Each pass should be thoroughly cleaned and visually inspected, though this is done by the welder and there is no expectation that the inspector inspect each pass. Control of electrodes, especially low hydrogen SMAW electrodes, must be maintained. In some cases, nondestructive testing may be performed at various stages during welding.

After the weld has been completed, the final size and location is checked to verify that it meets the plans and specifications of the contract. Visual inspection is performed, and nondestructive testing of the completed weld may be performed if required by the contract documents. If repairs are required, the inspection should include the repair work and reinspection of the repaired weld. The inspector responsible for the completed weld should place an identifying mark near the weld or on the piece. Written documentation of the weld's quality, including any noted significant discontinuities, should be prepared.

8.7 Nondestructive Testing

Several methods of nondestructive testing (NDT), also called nondestructive examination (NDE), may be used on a structural steel project.

The frequency and location of NDT must be stated in the contract documents. Little NDT is mandatory under the various steel construction codes except for fatigue applications and for certain types of joints in seismic applications.

The first common form of NDT is *visual testing* (VT). Most visual inspection is performed without the use of magnifiers. Magnifying glasses can be used to more closely examine areas that are suspected of cracks and other small, but potentially significant discontinuities. Adequate light and good visual acuity is necessary. Various weld gages are used to determine weld size and various other measurements.

An expanded form of visual inspection is *penetrant testing* (PT), also called dye penetrant or liquid penetrant testing. The weld surface and surrounding steel is thoroughly cleaned. A penetrating liquid dye is applied to the weld or steel surface and allowed time to penetrate cracks, pores, and other surface discontinuities. After an allotted time (dwell time), the penetrant is removed and a developer is applied to the surface. The developer draws the penetrant back to the surface of the weld or steel. The developer is of a color (usually white) that contrasts with the color of the dye in the penetrant. The inspector observes the dye in the developer, then removes the developer and dye to more closely inspect the surface visually. Some penetrant tests use an ultraviolet solution, rather than a dye, when a UV lamp is available. Penetrant testing can detect surface discontinuities only. Although PT is generally a visual technique, permanent records of discovered defects are frequently made with the use of digital photography.

Magnetic particle testing (MT) can be used to detect surface and slightly subsurface discontinuities. The general limit to the depth of inspection is around $^1/_8$ in (3 mm). Magnetism is induced into the region of the weld through the use of a yoke, on and near the surface of the steel. Magnetic particles such as fine iron particles are then applied to the surface of the steel. These particles are most commonly in the form of a dry powder, but may be in a liquid emulsion.

When cracks or other discontinuities are on or near the surface, the magnetic field is interrupted. In essence, the interruption has created two new magnetic poles in the steel, attracting the particles. The inspector then observes and interprets the position and nature of the accumulated particles, judging them to indicate a crack or other discontinuity on or near the surface. For best performance, the flux lines must flow perpendicular to the discontinuity. Therefore, the MT technician must rotate the yoke along the length of the weld to inspect for both longitudinal and transverse discontinuities. Although MT is generally a visual technique, permanent records of discovered defects are frequently made with the use of digital photography.

Ultrasonic testing (UT) is a very popular method of nondestructive testing. It is capable of testing weldments from approximately $^5/_{16}$ to 8 in (8 to 200 mm) in thickness. The most common method of testing uses a pulse-echo mode similar to radar or sonar. The control unit sends high-frequency electronic signals into a transducer made of piezoelectric material. The electrical energy is transformed by the transducer into vibration energy. The vibration is transmitted into the weldment through a coupling liquid. The vibration carries through the steel until a discontinuity or other interruption, such as an edge or end of the material, disrupts the vibration. The disruption reflects the ultrasound wave back toward the transducer. The return vibration is then converted back into electrical energy by the transducer, sending a signal to the display unit. The return signal's configuration, strength, and time delay are then interpreted by the testing technician.

The interpretation by the technician uses the height of the response signal to indicate severity. Using a calibration setup, the distance on the display unit from the initial pulse to the reflection is used to determine the distance from the transducer to the discontinuity. The operator can also manipulate the transducer in various patterns to determine a better understanding of the location, length, depth, orientation, and nature of the discontinuity.

AWS D1.1 Table 6.6 prescribes the testing procedures for butt, tee, and corner joints of various thicknesses. The search angle and faces to be used are given. Annex S of AWS D1.1 provides for alternative techniques for ultrasonic testing and the evaluation of weld discontinuities. With modern digital equipment, records of indications can be stored and later printed.

Radiographic testing (RT) is another method of NDT for welds in structural steel. RT is performed using either x-rays or gamma rays, sending energy into the steel weldment. Film is placed on the side of the weldment opposite the energy source. The steel and weld metal absorb energy, reducing the exposure of the film, but weld discontinuities allow more energy to expose the film. This increased exposure produces a darkened area on the film to be interpreted by the radiographer.

Radiographic testing is effective in steels up to approximately 9 in (230 mm) in thickness. X-ray machine capabilities depend upon the voltage setting of the machine, with 2000 kV required for an 8-in (200-mm) thickness. Gamma-ray machine capabilities depend upon the isotope used, usually cobalt-60 or iridium-192, but sometimes cesium-137. Exposure time and film selection are varied according to conditions and thicknesses. Image quality indicators (IQIs), either wire-type or hole penetrameters, are used to verify the sharpness and sensitivity of the film image, as well as to provide a measurement scale on the exposed film.

Because of the radiation exposure hazards and the time and equipment involved, radiographic testing is typically the most expensive of the methods previously mentioned. It also often fails to detect small cracks, small lack of fusion defects, and cracks and laminations that are oriented perpendicular to the energy source.

8.8 Weld Acceptance Criteria

The acceptance criteria to be used for the required weld quality are to be established by the engineer, as provided in AWS D1.1 Clause 1.4.1. Commonly, the quality and inspection criteria found in AWS D1.1 Clauses 5 and 6 are adopted. However, the engineer's specification of alternate criteria is permitted. Clause 6.8 of AWS D1.1 states that "The fundamental premise of the Code is to provide general stipulations applicable to most situations. Acceptance criteria for production welds different from those specified in the Code may be used for a particular application, provided they are suit ably documented by the proposer and approved by the Engineer." The Commentary to Clause 6.8 provides additional insights into the development and use of alternate acceptance criteria.

The visual acceptance criterion for welds is summarized in Table 6.1 of AWS D1.1. This table is broken down into three categories of connections: statically loaded nontubular, cyclically loaded nontubular, and tubular. These values also apply when penetrant testing (PT) and magnetic particle testing (MT) are used. Generally, cyclically loaded and tubular connections require a higher standard of quality than that required for statically loaded nontubular connections.

When ultrasonic testing is used, AWS D1.1 Table 6.2 is used for statically loaded nontubular connections and AWS D1.1 Table 6.3 is used for cyclically loaded nontubular connections. For tubular connections, use AWS D1.1 Clause 6.13.3. Alternately, the techniques of AWS D1.1 Annex S may be employed when approved by the engineer.

When radiographic testing is used, AWS D1.1 Figure 6.1 is used for statically loaded nontubular connections, Figure 6.4 for cyclically loaded nontubular tension connections, and Figure 6.5 for cyclically loaded nontubular compression connections. For tubular connections, AWS Clauses 6.12.3 and 6.18 apply.

Because many of the acceptance criteria found in AWS D1.1 are based upon what a qualified welder can provide, rather than the quality necessary for structural integrity, alternate acceptance criterion can be used to save both time and money. In addition, repairs to some welds with innocuous discontinuities may result in more damage to the material in the form of additional discontinuities, lower notch toughness, larger heat-affected zones, more distortion, and higher residual stresses.

Alternative acceptance criteria have been published by several organizations in various forms. In the United States, the Electric Power Research Institute has published "Visual Weld Acceptance Criteria," Document NP-5380, for use in reinspections of welds in existing nuclear power plant facilities. This weld acceptance criterion was accepted for use by the Nuclear Regulatory Commission. The Welding Research Council has published several WRC bulletins providing suggested criteria. The ASME *Boiler and Pressure Vessel Code,* Section IX, provides acceptance criteria for welds that can be also used for structural welds. The International Institute of Welding has published several documents providing suggested acceptance criteria, with considerable research documentation justifying the criteria. British Standards Institution document BS7910 "Guidance on Methods for Assessing the Acceptability of Flaws in Fusion Welded Structures," is one of the most thorough documents currently available.

8.9 Welding Inspector Certification Programs

Welding inspectors must be qualified to perform the work on the basis of the contract documents and the building code. Most commonly, the basis of inspector qualification is that of AWS D1.1 Clause 6.1.4.1. Under this provision, a welding inspector may be qualified by being an AWS Certified Welding Inspector, a welding inspector certified under the Canadian Welding Bureau, or "an individual who, by training or experience, or both, in metals fabrication, inspection and testing, is competent to perform inspection of the work." AWS D1.1 Clause 6.1.4.4 requires that the individual pass an eye examination, taken with or without corrective lenses. AWS B5.1 "Specification for the Qualification of Welding Inspectors" can also be used to establish the qualification of welding inspectors, requiring examinations and experience, but allowing for employer-based testing. The inspector's qualification to perform the inspection remains effective indefinitely, unless there is specific reason to question the inspector's abilities. In this case, the engineer or similar individual should evaluate the inspector's abilities and credentials.

The International Code Council offers certification for special inspectors in the areas of "Structural Steel and Bolting" and "Structural Welding."

For individuals performing only nondestructive testing work, the inspector need not be generally qualified for welding inspection.

However, the individual must be qualified using the provisions of the American Society for Nondestructive Testing "Recommended Practice No. SNT-TC-1A." This document provides recommendations for the training, experience level, and testing of NDT technicians. A suitable alternative to the "Recommended Practice," although not referenced in AWS D 1.1, is the ASNT CP-189 "Standard for Qualification and Certification of Nondestructive Testing Personnel."

These documents provide specific listings applicable to several areas of NDT, including

Radiographic testing

Magnetic particle testing

Ultrasonic testing

Penetrant testing

Visual testing

NDT technicians are placed into four categories. Formal definitions vary between the recommended practice and the standard. Using the definitions of the standard, the Level III technician has "the skills and knowledge to establish techniques; to interpret codes, standards and specifications; to designate the particular technique to be used; and to verify the adequacy of procedures." This individual is responsible for the training and testing of other NDT personnel in the individual's area of certification. The Level II technician has "the skills and knowledge to set up and calibrate equipment, to conduct tests, and to interpret, evaluate, and document results in accordance with procedures approved by an NDT Level III." The Level I technician has "the skills and knowledge to properly perform specific calibrations, specific tests, and with prior written approval of the Level III, perform specific interpretations and evaluations for acceptance or rejection and document the results." The trainee is a technician who works under the supervision of a Level II or III, and cannot independently conduct any tests or prepare reports of test results.

Chapter

9

Steel Deck Connections

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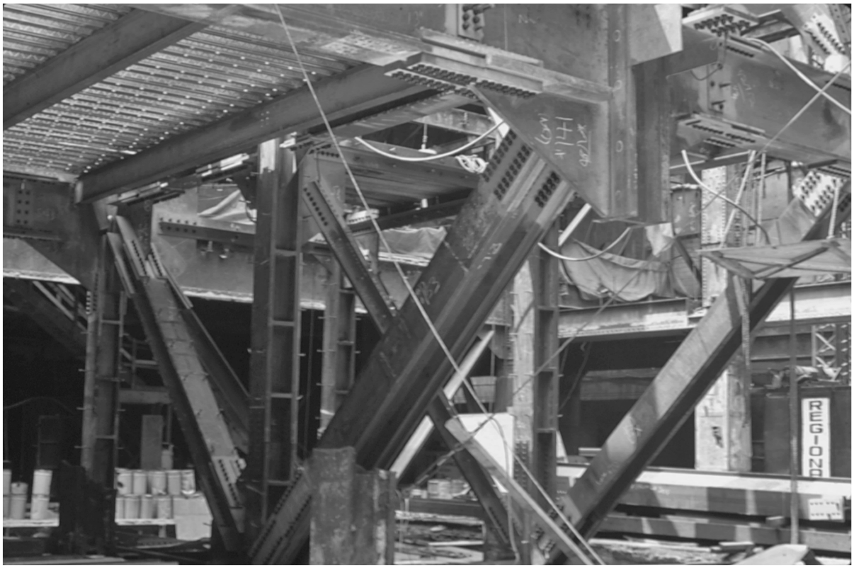
Fastening deck is an important design function which requires the attention of the design professional. Unlike structural steel, the fastening of steel deck has little or nothing to do with its fabrication so the deck supplier has no responsibility to choose the type of fastener or the spacing. However, the deck supplier, or the Steel Deck Institute (SDI), can aid the designer by providing information that can be helpful in the selection process.

For construction purposes the deck is almost always used as a working platform. It is, therefore, quite important that the deck be quickly and adequately attached as it is placed. Additionally, the fastened deck acts to stabilize joists and brace beams. Although the construction process is usually not part of the design, safety of the working platform is obviously important.

The factors that most affect the fastening are the anticipated wind and earthquake loads. These cause both horizontal (diaphragm) and vertical (uplift) forces to be applied to the fasteners and, as a result, are of most interest to designers. In the case of wind, shear and tension interaction exists and interaction equations are provided in the SDI *Diaphragm Design Manual*, Third Edition. Some Underwriters Laboratories' fire-rated constructions also specify fastener types and spacing, and must be consulted.



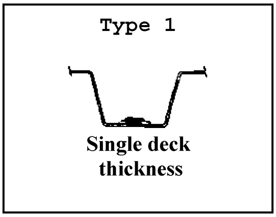
(*Courtesy of The Steel Institute of New York.*)

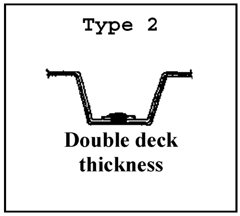
Shear and uplift strengths are the parameters most needed. Table 9.1 shows the ultimate tensile strength of arc spot (puddle) welds through steel deck. Three types of welds are illustrated in Table 9.1: Type 1 is through a single deck thickness, Type 2 is at a deck end lap or through cellular deck and is through two thicknesses of steel, and Type 3 is at a deck edge (side) lap and its lower values are the result of the eccentric loading at the edge—a 0.7 multiplier is applied to the Type 1 value. An end member prying factor of 2 is also required in design for welds but this is only critical at single spans since an interior support tributary area is normally twice that at the end of the member. The minimum recommended diameter is $\frac{1}{2}$ in (13 mm) but $\frac{5}{8}$ in (16 mm) is common.

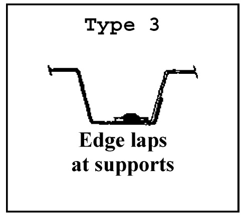
Table 9.2 shows the nominal (ultimate) shear strength of puddle welds. These are to be used for diaphragm loads. The resistance and safety factors for diaphragms are based on the AISI *North American Specification for the Design of Cold-Formed Steel Structural Members, 2007 Edition*. Stress increase for temporary loading is not allowed. The diaphragm factors are system and repeating member factors. When isolated fasteners are used to resist shear, use the individual factors presented in the North American Specification. A quality arc spot weld should have at least 75% of the perimeter attaching the deck to the structural steel. A quality control procedure is shown in Fig. 9.1.

Weld washers are only recommended for attaching deck to the structural frame or bar joists when the deck steel is less than 0.028 in (0.71 mm) thick. The purpose of the weld washers is to provide a heat sink and keep the

TABLE 9.1 Tensile Strength of Arc Spot (Puddle) Welds, lb.*







		Visible Weld Diameter (in)								
		Type 1			Type 2			Type 3		
Steel	Gage	0.5	0.625	0.75	0.5	0.625	0.75	0.5	0.625	0.75
A653 SS 33	22	930	1180	1420	1740	2240	2730	650	820	1000
F_y = 33 ksi	20	1110	1410	1710	2050	2650	3250	780	990	1200
F_u = 45 ksi	18	1440	1830	2230	2030	3360	4160	1010	1280	1560
	16	1760	2260	2760	1370	3140	4460	1230	1580	1930
A653 SS 40	22	1150	1460	1770	2160	2780	3390	810	1020	1240
F_y = 40 ksi	20	1380	1750	2130	2550	3300	4040	970	1230	1490
F_u = 55 ksi	18	1780	2280	2770	2030	4110	4540	1250	1590	1940
	16	2190	2810	3430	1370	3140	4540	1530	1970	2400
A653 SS 50	22	1220	1540	1870	2290	2930	3580	850	1080	1310
F_y = 50 ksi	20	1460	1850	2250	2700	3480	4060	1020	1300	1570
F_u = 65 ksi	18	1890	2410	2930	2030	4060	4060	1320	1680	2050
	16	2310	2970	3630	1370	3140	4060	1620	2080	2540
A1008 SS 33	22	1130	1430	1730	2110	2710	3310	790	1000	1210
F_y = 33 ksi	20	1350	1710	2080	2490	3220	3950	950	1200	1450
F_u = 48 ksi	18	1740	2220	2710	2030	4080	5050	1220	1560	1890
	16	2140	2750	3350	1370	3140	5080	1500	1920	2350
A1008 SS 40	22	980	1240	1490	1830	2350	2870	680	860	1050
F_y = 40 ksi	20	1170	1480	1800	2160	2790	3410	820	1040	1260
F_u = 52 ksi	18	1510	1920	2340	2030	3530	4060	1060	1350	1640
	16	1850	2380	2900	1370	3140	4060	1300	1660	2030

Table resistance values are nominal values in pounds. Factors must be applied to determine service loads. For deck:

ASD safety factor (Ω) = 2.5 LRFD factor (ϕ) = 0.6

The basis is the AISI Standard *North American Specification for the Design of Cold-Formed Structural Members.*

Roof deck is generally specified to ASTM A653 SS 33 (galvanized) or A1008 SS 33 (no galvanize but painted). SS indicates structural steel grade. SS 80 is used but design defaults to F_y = 60 ksi and F_u = 62 ksi.

Follow AWS D1.3 procedures for arc spot (puddle) welds. Minimum electrode strength of 60 ksi is required. The minimum required clear distance between the weld edge and the end of the deck is the visible diameter. Welds larger than 3/4 in are possible but are rarely used.

Follow the local codes for design loads, load combinations, and load factors. If no code exists, use ASCE 7.

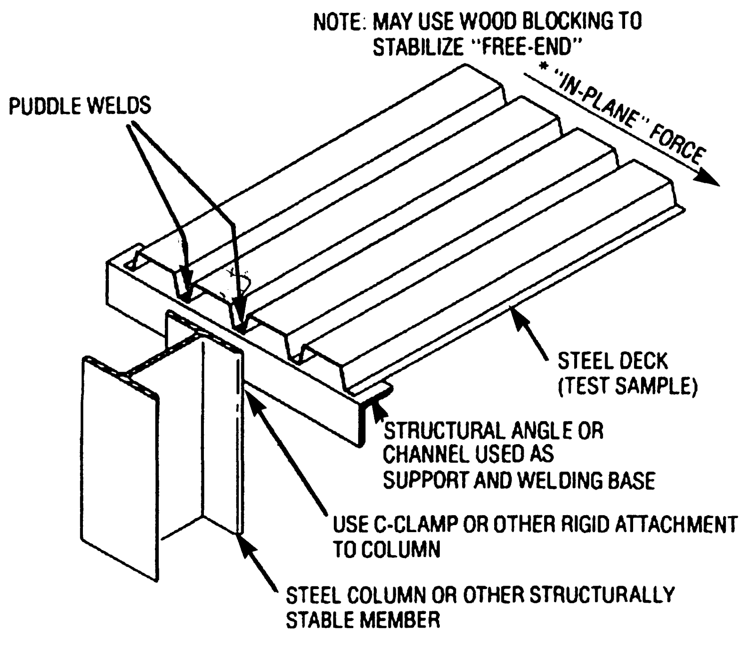
**Courtesy of CMC Joist & Deck, Manufacturers of United Steel Deck Products.*

TABLE 9.2 Weld Shear Strengths, lb (for Diaphragm Calculations)*

These values are based on the Steel Deck Institute *Diaphragm Design Manual*, 3d ed. The values are based on $F_y = 33$ ksi, $F_u = 45$ ksi, $F_{xx} = 70$ ksi. Resistance is linear with F_u but the table values conservatively may be used.

Metal thickness	Visible weld diameter, in		Factors for Diaphragms		
	0.625	0.75	Load type or combinations including	Safety factor (ASD) Ω	Resistance factor (LRFD) ϕ
0.0295	1739	2104			
0.0358	2088	2531			
0.0418	2413	2931	Seismic	3.00	0.55
0.0474	2710	3297	Wind	2.35	0.70
0.0598	3346	4086	All others	2.50	0.60

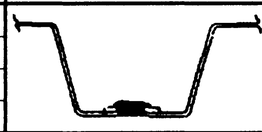
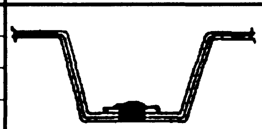
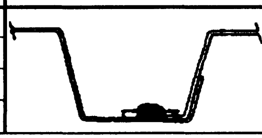
Courtesy of the Steel Deck Institute.



A preliminary check for welding machine settings and operator qualifications can be made through a simple field test by placing a pair of welds in adjacent valleys at one end of a panel. The opposite end of the panel can then be rotated, which places the welds in shear. Separation leaving no apparent external weld perimeter distresses, but occurring at the sheet-to-structure plane, may indicate insufficient welding time and poor fusion with the substrate. Failure around the external weld perimeter, showing distress within the panel but with the weld still attached to the substrate, would indicate a higher quality weld.

Figure 9.1 Weld quality control check. (*Courtesy of the Steel Deck Institute.*)

TABLE 9.3 Weld Washer Strengths

Case	Gage	Pn, Uplift (Tensile) Values, lbs[1]		Shear, lbs[2]	Profile
1	28	0.0149	1390	1200	
	26	0.0179	1430	1550	
	24	0.0239	1520	2350	
	23	0.0269	1560	2830	
2	28	0.0149	1590	1200	
	26	0.0179	1670	1550	
	24	0.0239	1840	2350	
	23	0.0269	1780	2830	
3	28	0.0149	960	1200	
	26	0.0179	990	1550	
	24	0.0239	1050	2350	
	23	0.0269	1090	2830	

(1) A suggested safety factor (ASD) is 2.5; the recommended ϕ factor (LRFD) is 0.60.
(2) A recommended safety factor (ASD) is 2.75; the recommended ϕ factor (LRFD) is 0.55.
The table is based on typical form deck material (F_y = 80 ksi); a 70 ksi electrode strength was used. Washers are 16 gage.

weld burn from consuming too much of the thin steel. The weld washer then forms a "head" on the weld button and provides the uplift and shear strengths as shown in Table 9.3. The 0.7 prying multiplier is applied at case 3. Common weld washers furnished by deck manufacturers are made of 16 gage material [0.057 in (1.44 mm)] and have a $^3/_8$-in (10-mm)-diameter hole. The weld should slightly overfill the hole to produce a visible weld diameter of approximately $^1/_2$ in (13 mm). Figure 9.2 shows the patterns and pattern nomenclature of deck-to-frame connections.

Self-drilling screws are frequently used as deck-to-frame attachments. These are installed with an electric screw gun that has a clutch and a depth-limiting nose piece to prevent over-torque. Screws are #12s or #14s ($^1/_4$ in) with the drill point selected to drill through the total metal thickness of deck and beam (or joist) flange. Uplift (pullover and pullout) values are shown in Table 9.4. The lesser value of P_{nov} and P_{not} is used in design. Screws are preferred when fastening to light gage framing that is thinner than 10 gage. Self-drilling screws are available for the special application of steel deck to wood framing. Consult the screw manufacturer for proper selection and specification of fasteners. Corrosion of screws in timber due to moisture and salt preservatives must be considered.

Other excellent deck-fastening methods have been developed which compete with traditional welds and screws. These fastening methods use powder or air pressure to drive pins through the deck into structural

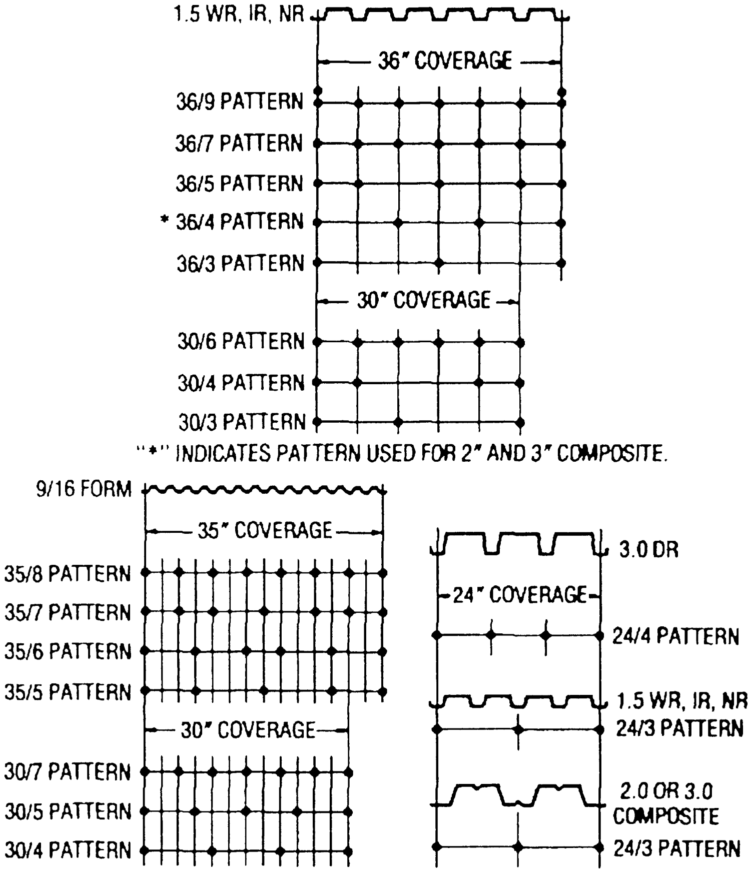


Figure 9.2 Frame connection layouts. (*Courtesy of the Steel Deck Institute.*)

steel. These fasteners are proprietary and not generically covered in the AISI specification. Strength values and diaphragm tables are published by the manufacturers of these products and they also provide technical assistance for designers. The Steel Deck Institute advises that, "No substitution of fastener type or pattern should be made without the approval of the designer." Fastener manufacturers can provide the data needed to make substitutions using their products.

Shear studs can be welded through the deck into the steel framing with an automatic stud "gun." The primary function of the studs is to make the beam act compositely with concrete but they also act to fasten the deck to the frame. Studs also increase the composite deck slab capacity. Shear studs can be welded through two well-mated thicknesses of steel such as cellular deck. But, for deck heavier than 16 gage, consultation with the stud manufacturer is advised. Welding time and settings are dependent on: deck coating, steel thickness, and ambient conditions.

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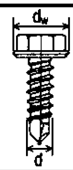
TABLE 9.4 Uplift Values for Screwed Deck

SCREW DATA			
Screw Size	d dia.	d_w nom. head dia.	Avg. tested tensile strength, kips
10	0.190	0.415 or 0.400	2.56
12	0.210	0.430 or 0.400	3.62
¼	0.250	0.480 or 0.520	4.81



Pull Out Values, kips

$P_{not} = 0.85t_2dF_{u2}$; Steel thickness = t_2, F_{u2} = tensile strength of support										
Screw	Gage									
	1/4"	3/16"	10	1/8"	12	14	16	18	20	22
#10	2.34	1.76	0.98	1.17	0.76	0.54	0.43	0.34	0.26	0.21
#12	2.66	2.00	1.11	1.33	0.86	0.62	0.49	0.39	0.30	0.24
1/4"	3.08	2.31	1.29	1.54	1.00	0.71	0.57	0.45	0.34	0.28

The table pullout strengths kips, are based on F_u = 45 for 10, 12 thru 22 gage, and 58 ksi for 1/4", 3/16", and 1/8". The thread must penetrate the full thickness.

Pull Over Values, kips

$P_{nov} = 1.5t_1d_wF_{u1}$; $d_w < 0.50$"; Sheet thickness = t_1, F_{u1} = tensile strength of sheet							
d_w	Gage						
	16	18	20	22	24	26	28
0.400	1.61	1.28	0.97	0.80	0.89	0.67	0.55
0.415	1.68	1.33	1.00	0.83	0.92	0.69	0.58
0.430	1.74	1.38	1.04	0.86	0.96	0.72	0.60
0.480	1.94	1.54	1.16	0.96	1.07	0.80	0.67
0.500	2.02	1.60	1.21	1.00	1.11	0.83	0.69

The table pull over strengths kips, are based on F_u = 45 ksi for 16 thru 22 gage, and 62 ksi for 24 thru 28 gage.
The safety factor, Ω, for pull over (ASD) is 3. The ϕ factor (LRFD) is 0.5.

Deck can be screwed to structural steel, SJI joists or light gage steel framing. The lowest steel strength was used to produce the tabulated values. For joists and structural steel, a tensile strength (F_u) of 58 ksi was used which is the lowest value for A36 steel. For gage supports, F_u =45 ksi was used which is the lowest provided in ASTM A653 for Structural Steel grade 33. Deck material furnished in gages 24, 26 28 are usually grade 80 steel which uses a tensile strength (F_u) of 62 ksi as limited by the AISI specifications. Either pull out of the screw or pullover of the deck will normally control. The values are based on the equations provided by the AISI Specifications. These specifications do not require prying at side-laps or ends. A safety factor of 3 is applied for ASD & a ϕ factor of 0.5 is applied for LRFD. If it is known that the tensile strength of the support steel or the sheet steel is greater than those used for the tables, the tabulated screw ultimate strengths may be increased by a straight line ratio.

Although research is ongoing, paint is not recommended on the beam surface receiving studs. The American Welding Society (AWS D1.1) provides a quality control check for welded studs.

Deck-to-deck connections at side laps are sometimes called "stitch connections." Screws, welds, and button punches are the usual ways to accomplish the connection. The primary purpose of side-lap attachments is to let adjacent sheets help in sharing vertical and horizontal loads.

Stitch screws are usually of the self-drilling type; #8s through #14 ($^1/_4$ in) diameter can be used but screws smaller than #10s diameter are not recommended. The installer must be sure that the underlying sheet is drawn tightly against the top sheet and that adequate edge dimension is maintained—the normal rules are 1.5 times the screw shank diameter measured to the center of the screw and as required to develop shear when edge is measured parallel to the line of force. Again, as when screws are used as the frame attachment, special screw-driving guns are used to prevent over-torque.

Manual button punching of side laps requires a special crimping tool. Button punching requires the worker to adjust his or her weight so the top of the deck stays level across the joint. Since the quality of the button punch attachment depends on the strength and care of the tool operator, it is important that a consistent method be developed. Automatic power-driven crimping devices are rarely seen on deck jobs but should not be ruled out as a fastening method. Some manufacturers do provide proprietary crimping tools and can provide test based diaphragm load tables using these connections.

Good metal-to-metal contact is necessary for sidelap welds. Burn holes are the rule rather than the exception and an inspector should not be surprised to see them in the deck. The weld develops its strength by holding around the perimeter and a good weld will have 75% or more of its perimeter working. On occasion, side-lap welds will be specified for deck that has the button punchable side-lap arrangement (see Fig. 9.3 for comments

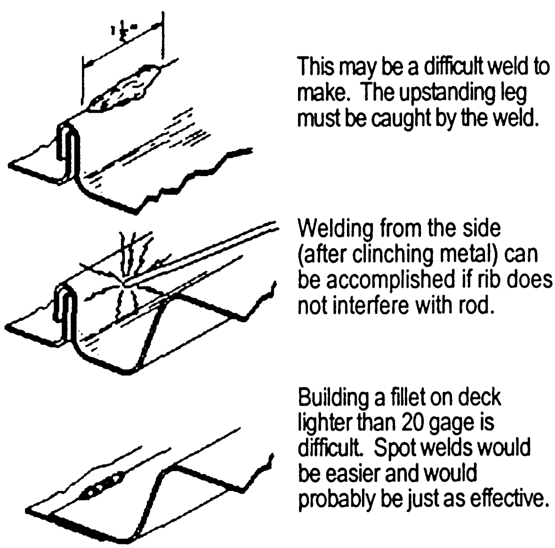


Figure 9.3 Sheet-to-sheet welds between supports. (*Courtesy of the Steel Deck Institute.*)

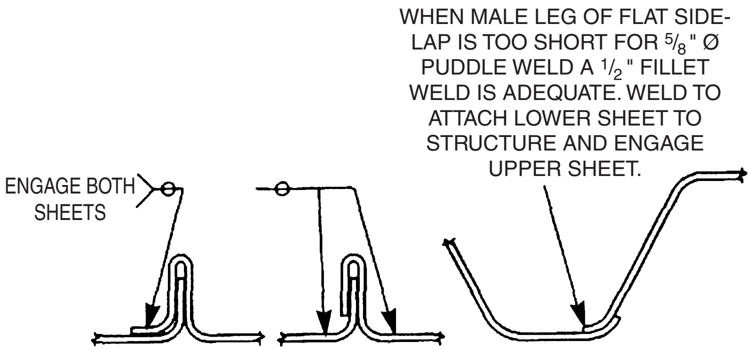


Figure 9.4 Side-lap welds at supports. (*Courtesy of the Steel Deck Institute.*)

on this subject; see Fig. 9.4 for welding these deck units to the frame). Welding side laps is not recommended for a 22-gage deck (0.028 in) or lighter. Weld washers should never be used at side laps between supports. The SDI recommends that side laps be connected at a maximum spacing of 36 in (1 m) for deck spans greater than 5 ft (1.5 m). This minimum spacing could be increased to enhance diaphragm values. Edge fasteners parallel with the deck span and over supports are recommended. Supports that are parallel to the deck span and between support beams are recommended at roof perimeters or shear walls. This allows edge fasteners. The edge fastener spacing at these parallel supports should match the deck side-lap fastener spacing.

Accessories attached to the deck are welded, screwed, pop riveted, or (rarely) glued. Usually the choice is left to the erector and many times is simply the result of the tools available at the time. The importance of fastening accessories can be either structural or architectural, and the designer may need to become involved. For instance, the attachment of reinforcement around penetrations, and the fastening of pour stops, may have a great deal to do with the expected performance of the accessory and care must be taken to see that sufficient attachment is done. If the deck is to be exposed to view, then architectural considerations might be of concern and the fasteners may be selected accordingly. Button punched side laps are often specified at exposed interlocking or cellular deck side laps.

Frequently the expression “tack welding” is used to describe attachment of accessories to deck or to structural steel. A *tack weld* is defined by the AWS as “a weld made to hold parts of a weldment in proper alignment until the final welds are made.” The term, when applied to accessories, means a weld of unspecified strength or size simply used to hold the accessory securely in its proper position. When floor deck accessories are tack-welded, the concrete is usually the medium that will hold the parts in their final place. The accessories shown in Fig. 9.5 can be tack-welded or screwed as is appropriate. The one exception is the

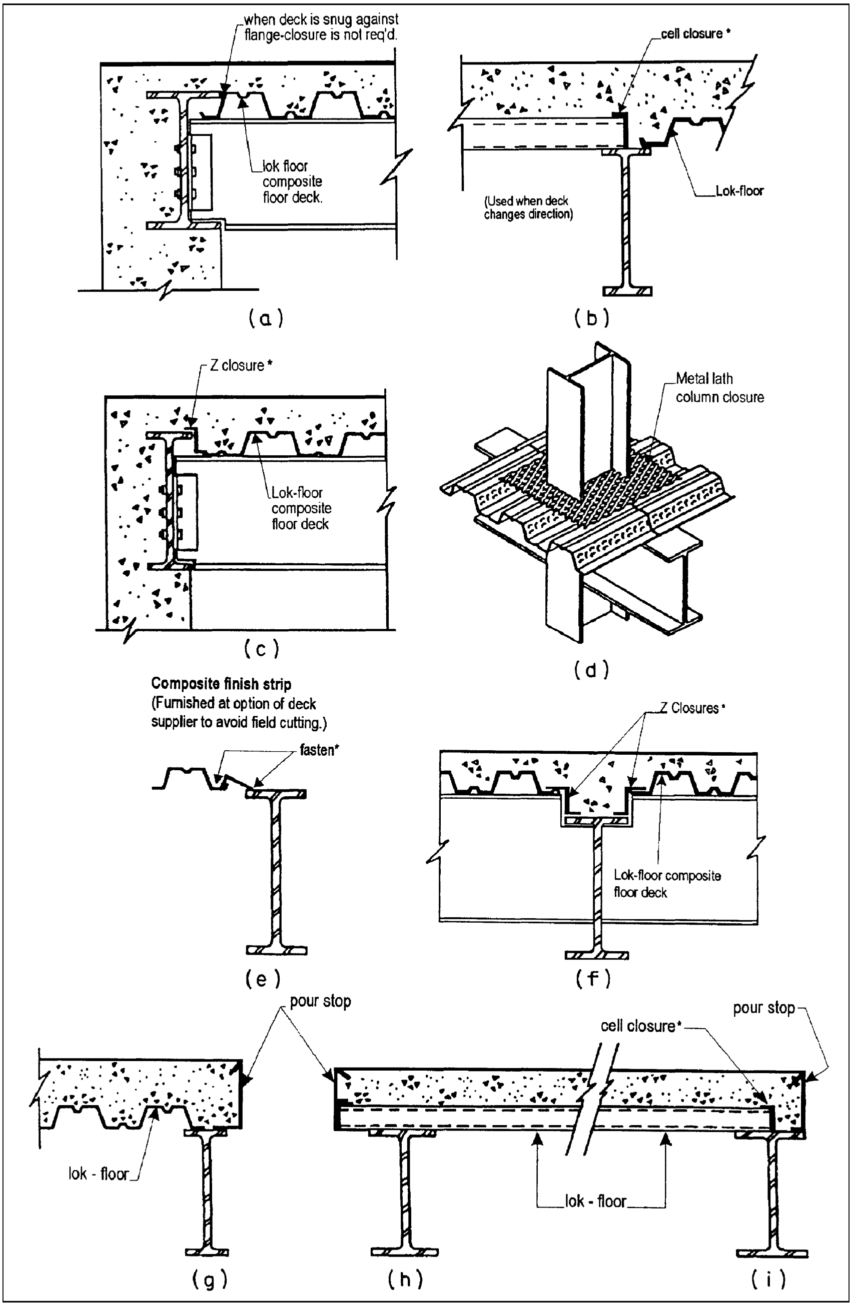


Figure 9.5 Fastening floor deck accessories. (*Courtesy of CMC Joist & Deck, Manufacturers of United Steel Deck Products.*)
*Attach closures to deck or supports using #10 screws (minimum) or 1-in fillet welds at a maximum spacing of 24 in on centers. Welds are commonly used at supports.

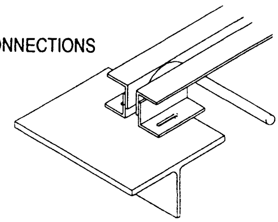
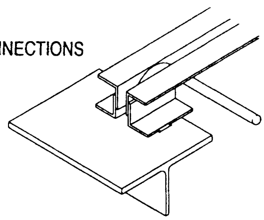
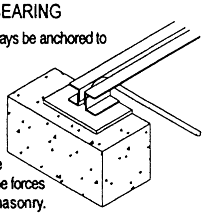
TYPE	K SERIES	LH/DLH SERIES
BOLTED CONNECTIONS 	Slotted holes in bearing plates are furnished whenever bolted connections are required. Bolts ($1/2$-in diameter) are not furnished by the joist manufacturer. Minimum bearing on structural steel supports is $2\frac{1}{2}$-in.	Slotted holes in bearing plates are furnished whenever bolted connections are required. Bolts ($3/4$-in diameter) are not furnished by the joist manufacturer. Minimum Bearing on structural steel supports is 4-in.
WELDED CONNECTIONS 	Ends of K Series joists are normally anchored by two 1/8-in fillet welds 1-in long. Minimum bearing on structural steel supports is $2\frac{1}{2}$-in.	Ends of LH/DLH Series joists are normally anchored by two 1/4-in fillet welds 2-in long. Minimum bearing on structurtal steel supports is 4-in.
TYPICAL MASONRY BEARING The setting plates should always be anchored to the masonry wall. The setting plate must be located not more than ½" from the face of the wall. The design professional must design the bearing plate and must take into account the forces acting on the concrete or masonry. 	Minimum bearing is 4-in.	Minimum bearing is 6-in.

Figure 9.6 Joist bearing details.

case of pour stops; the SDI calls for 1-in fillet welds at 12-in oc to the structural steel.

Some additional details on steel joist bearing and connections are shown in Figs. 9.6 and 9.7.

Composite beam details showing metal deck connected to steel beams are shown in Fig. 9.8. Table 9.5 presents the $3/4$-in diameter shear stud values in deck. Additional details of commonly used metal deck constructions are shown in Figs. 9.9 through 9.12. Industry generic details are available in the SDI publication, "Standard Practice Details."

Safety provisions mandated by OSHA in the latest edition of *29 CFR Part 1926 Subpart R* generally require that openings be "decked over" until the trade requiring the opening is ready to fill the opening. Unless directed otherwise by a site-specific erection plan, details should be consistent with this provision.

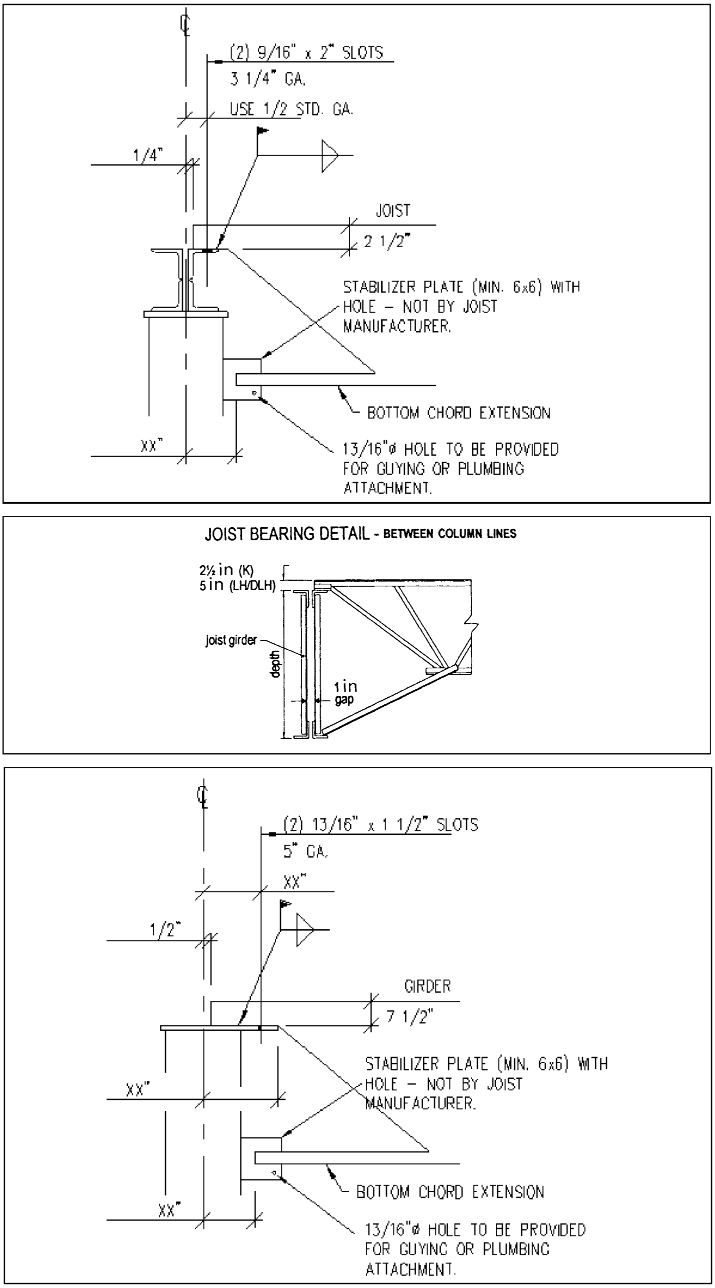
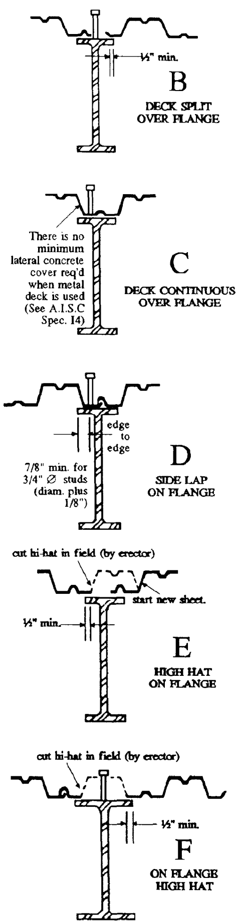
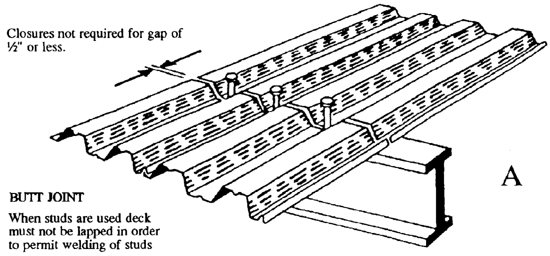


Figure 9.7 Joist bearing on joist girders.

GIRDER DETAILS





Special deck arrangements-when shear studs are present.

DETAILS A THROUGH F ARE APPLICABLE ONLY WHEN SHEAR STUDS ARE PRESENT.

DETAILS A THROUGH F MAY BE USED AT THE OPTION OF THE DECK SUPPLIER TO SATISFY VARIOUS DESIGN CASES.

All LOK-FLOOR is furnished in 24 inch or 36 inch widths. Any cutting that is required shall be done in the field by the deck erector.

Lateral and longitudinal spacing is controlled by sec. I5 A.I.S.C. Manual of Steel Construction - for location of stud to edge of flange see Structural Welding Code AWS 7.4.5.

DECK RIB RATIOS	
Deck Type	w/h
1.5" B - Lok	1.5
1.5" Inverted B - Lok	2.5
1.5" Lok - Floor	3.85
2" Lok - Floor	3.0
3" Lok - Floor	2.0

STUDS ARE ¾" DIAMETER			
Concrete*		Stud Strength	
		LRFD	ASD
f'_c, ksi	Weight, pcf	(Q_n, kips)	(Q_n, kips)
3.0	115	17.7	9.9
3.0	145	21.0	11.5
3.5	115	19.8	10.8
3.5	145	23.6	12.5
4.0	115	21.9	11.4
4.0	145	26.1	13.3

* 145 pcf concrete made with ASTM C33 aggregates. 115 pcf concrete made with ASTM C330 aggregates.

N = Number of studs per rib.
H = Length of stud.
h = Height of rib.
w = Average width of rib.

$$\text{Rib Coefficient} = \frac{0.85}{\sqrt{N}}\left(\frac{w}{h}\right)\left(\frac{H}{h} - 1.0\right) \le 1.0$$

For best construction of composite beams place studs in the portion of the deck rib closest to the beam end.

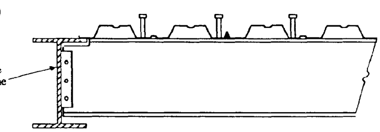


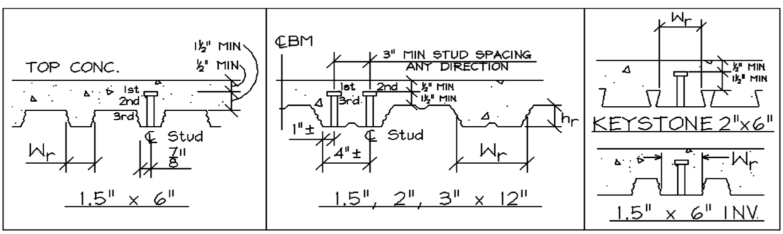
Figure 9.8 Composite beam details.

TABLE 9.5 Shear Stud Strength

Special Note: Section I3.2c of the *2005 ANSI/AISC Specification for Structural Steel Buildings* in the *AISC ASD/LRFD Steel Construction Manual, 13th Edition* specifies new strength criteria for studs in composite beams with formed deck. This table supersedes the values in the SDI Standard Practice Details, May 2001 and the revisions of April 2003.

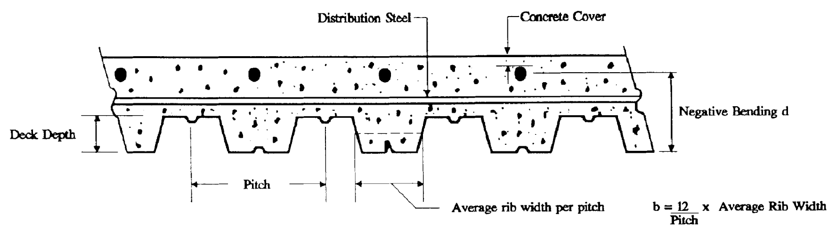
Q_n in Steel Deck - LRFD
3/4 in ϕ Shear Stud's Average Nominal Shear Strength/Stud in a Deck Corrugation, kips

Profile	w_r in	Concrete density, pcf	Studs per corr.	Perpendicular to beam			Parallel to beam		
				f′c Concrete compressive strength, ksi					
				3.0	3.5	4.0	3.0	3.5	4.0
Solid concrete	NA	145	NA	21.0	23.6	26.1	21.0	23.6	26.1
		115		17.7	19.8	21.9	17.7	19.8	21.9
1.5 × 6 deck	2.125	145	1	17.2			18.3		
			2	14.6					
			3	12.1					
		115	1	17.2			17.7	18.3	18.3
			2	14.6					
			3	12.1					
1.5, 2, 3 × 12	6	145	1	17.2			21.0	21.5	21.5
			2	16.5					
Keystone	4.6		3	13.1					
		115	1	17.2			17.7	19.8	21.5
1.5 × 6 inverted	3.75		2	16.5					
			3	13.1					



1. Preferred (strong) stud location is closest to beam ends in each corrugation with studs.
2. Table assumes that the first stud will be located in the weak location, the second in the strong location, and the third in the weak location. All studs through 1.5- × 6-in deck are in the weak location.
3. w_r = average width of rib = 2 in. When deck is parallel to beam, w_r = 5 in for two studs across the corrugation. For multiple studs in Keystone Deck or 1.5- × 6-in inverted, split the deck when parallel to beam and see values when perpendicular to beam.
4. h_r = height of rib = 3 in.
5. Density = 145 PCF conforms to ASTM C33; 115 PCF conforms to ASTM C330.
6. Studs conform to ANSI/AWS D1.1 with F_u = 65 ksi.
7. When deck is parallel to the beam: (a) The minimum center-to-center spacing of studs installed along the beam is 4½ in. (b) When w_r is wide enough, we suggest that studs be staggered either side of the corrugation. (c) Deck may be split over beams. (d) When studs are side-by-side, the minimum transverse spacing is 3 in.
8. The maximum center-to-center spacing of studs shall neither exceed 8 times total slab thickness nor 36 in.
9. Studs of lesser diameter are allowed and shear values and minimum spacing are reduced.

Courtesy of the Steel Deck Institute.



DECK SECTION	PITCH	AVERAGE RIB WIDTH	b WIDTH for NEGATIVE BENDING
B-LOK	6"	2.25"	4.5"
INV B-LOK	6"	3.75"	7.5"
LOK FLOOR	12"	6"	6"

Use Standard concrete design procedures as per ACI.

1.) Allowable bending stress of 20 ksi with loading of concrete + deck + 20 psf or concrete + deck + 150 lb. concentrated load, whichever is worse.

2.) Allowable deflection of free edge (based on fixed end cantilever) of 1/120 of cantilever span under loading of concrete + deck.

3.) Bearing width of 3½" assumed for web crippling check - concrete + deck + 20 psf over cantilever and adjacent span: if width is less than 3½" check with Summit, New Jersey office.

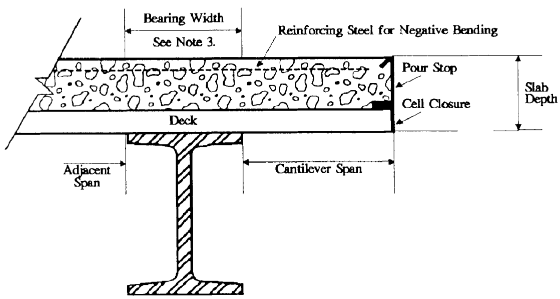


Figure 9.9 Negative bending information. LRFD methods are allowed and a deflection limit of $\frac{1}{90}$ has been adopted by industry.

FLOOR DECK CANTILEVERS

NORMAL WEIGHT CONCRETE (150 PCF)

	UNITED STEEL DECK, INC. DECK PROFILE															
	B-LOK				1.5 LOK-FLOOR				2.0 LOK-FLOOR				3.0 LOK-FLOOR			
Slab Depth	22	20	18	16	22	20	18	16	22	20	18	16	22	20	18	16
4.00"	1'11"	2'3"	2'10"	3'4"	1'11"	2'4"	3'0"	3'6"								
4.50"	1'10"	2'2"	2'9'	3'3"	1'10"	2'3"	2'10"	3'4"	2'6"	2'11"	3'8"	4'3"				
5.00"	1'10"	2'2"	2'8"	3'2"	1'10"	2'3"	2'9"	3'3"	2'5"	2'10"	3'6"	4'1"	3'8"	4'3"	5'3"	6'0"
5.50"	1'9"	2'1"	2'7"	3'0"	1'9"	2'2"	2'9"	3'2"	2'4"	2'9"	3'5"	4'0"	3'7"	4'1"	5'0"	5'9"
6.00"	1'9"	2'0"	2'6"	2'11"	1'9"	2'1"	2'8"	3'1"	2'3"	2'8"	3'4"	3'10"	3'5"	3'11"	4'10"	5'7"
6.50"	1'8'	2'0"	2'6"	2'11"	1'9"	2'1"	2'7"	3'0"	2'3"	2'8"	3'3"	3'9"	3'4"	3'10"	4'8"	5'5"
7.00"	1'8"	1'11"	2'5"	2'10"	1'8"	2'0"	2'6"	2'11"	2'2"	2'7"	3'2"	3'8"	3'3"	3'9"	4'6"	5'3"
7.50"	1'8"	1'11"	2'4"	2'9"	1'8"	2'0"	2'6"	2'10"	2'2"	2'6"	3'1"	3'7"	3'2"	3'8"	4'5"	5'1"
8.00"	1'7"	1'11"	2'4"	2'8"	1'7"	1'11"	2'5"	2'10"	2'1"	2'5"	3'0"	3'6"	3'1"	3'6"	4'3"	4'11"

Figure 9.10 Floor deck cantilevers. These values are dependent on the back span. Do not walk on the deck until properly fastened to supports. Side laps are to be fastened at 12-in oc at the cantilever.

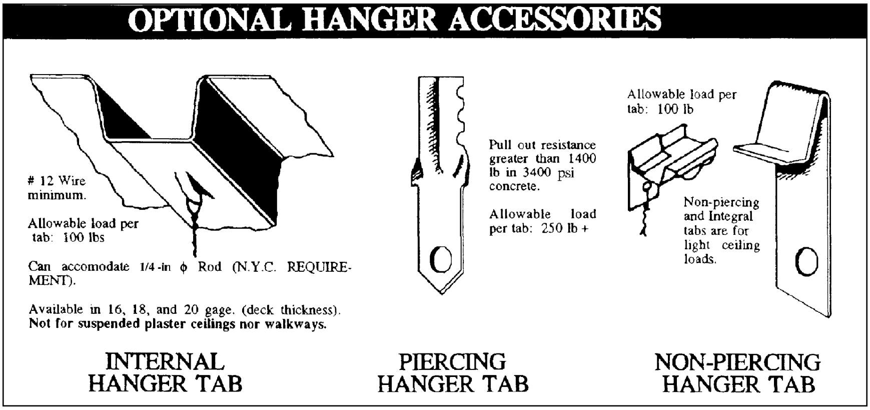
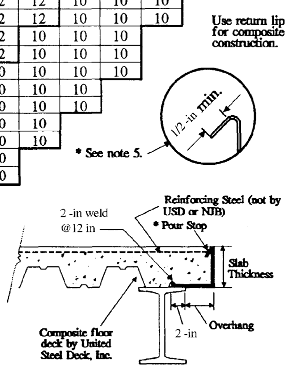


Figure 9.11 Optional hanger accessories.

SLAB DEPTH (inches)	OVERHANG (inches) POUR STOP TYPES 0	1	2	3	4	5	6	7	8	9	10	11	12
4.00	20	20	20	20	18	18	16	14	12	12	12	10	10
4.25	20	20	20	18	18	16	16	14	12	12	12	10	10
4.50	20	20	20	18	18	16	16	14	12	12	12	10	10
4.75	20	20	18	18	16	16	14	14	12	12	10	10	10
5.00	20	20	18	18	16	16	14	14	12	12	10	10	
5.25	20	18	18	16	16	14	14	12	12	12	10	10	
5.50	20	18	18	16	16	14	14	12	12	12	10	10	
5.75	20	18	16	16	14	14	12	12	12	12	10	10	
6.00	18	18	16	16	14	14	12	12	12	10	10	10	
6.25	18	18	16	14	14	12	12	12	12	10	10		
6.50	18	16	16	14	14	12	12	12	12	10	10		
6.75	18	16	14	14	14	12	12	12	10	10	10		
7.00	16	16	14	14	12	12	12	12	10	10	10		
7.25	16	16	14	14	12	12	12	10	10	10			
7.50	16	14	14	12	12	12	12	10	10	10			
7.75	16	14	14	12	12	12	10	10	10	10			
8.00	14	14	12	12	12	12	10	10	10				
8.25	14	14	12	12	12	10	10	10	10				
8.50	14	12	12	12	12	10	10	10					
8.75	14	12	12	12	12	10	10	10					
9.00	14	12	12	12	10	10	10						
9.25	12	12	12	12	10	10	10						
9.50	12	12	12	10	10	10							
9.75	12	12	12	10	10	10							
10.00	12	12	10	10	10	10							
10.25	12	12	10	10	10								
10.50	12	12	10	10	10								
10.75	12	10	10	10									
11.00	12	10	10	10									
11.25	12	10	10										
11.50	10	10	10										
11.75	10	10											
12.00	10	10											



TYPES	DESIGN THICKNESS
20	0.0358
18	0.0474
16	0.0598
14	0.0747
12	0.1046
10	0.1345

This Selection Chart is based on following criteria:

1. Normal weight concrete (150 pcf).

2. Horizontal and vertical deflection is limited to 1/4-in maximum for concrete dead load.

3. Design stress is limited to 20 ksi for concrete dead load temporarily increased by one-third for the construction live load of 20 pcf.

4. Pour Stop Selection Chart does not consider the effect of the performance, deflection, or rotation of the pour stop support which may include both the supporting composite deck and/or the frame.

5. Vertical leg return lip is recommended.

6. This selection is not meant to replace the judgement of experienced Structural Engineers and shall be considered as a reference only.

Figure 9.12 Pour stop selection chart. Industry recommends a minimum weld of 1 in at 12 in oc. This detail is slightly more restrictive.

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Chapter

10

Connections to Composite Members

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(*Courtesy of The Steel Institute of New York.*)

10.1 Introduction

The combined use of steel and concrete to form composite structures has been used widely. The introduction of new composite building systems has allowed the design and construction of more efficient mid- and high-rise composite buildings. In most composite building systems, the main problem facing designers has been the selection of an appropriate and economical connection.

This chapter provides suggestions for connection details for three types of composite structure elements: (1) connection details for connecting coupling steel beams to reinforced concrete shear walls (Sec. 10.3), (2) joint details for connecting steel beams to reinforced concrete columns (Sec. 10.4), and (3) connection details for attaching steel beams to rectangular and circular concrete-filled steel-tube columns (Sec. 10.5).

10.2 General Design Considerations

The design of connections, in general, requires consideration of stiffness, strength, stability, serviceability, and cyclic behavior. Following is a brief discussion of each of these items.

10.2.1 Strength and stiffness

When connections are subjected to applied moment, they will cause rotation at the member end. For instance, if a beam is attached to a column using top and seat angles, the applied moment to the beam end generated by vertical or lateral loads will cause the beam end to rotate with respect to the column face. The amount of this rotation is dependent on the stiffness of the connected elements. Experimental results indicate that all connections exhibit some level of rotation and, therefore, one could argue that all connections are semirigid. For design purposes, however, design manuals divide connections into three categories: (1) connections that exhibit relatively large end rotations (simple connections), (2) connections that result in very small rotation (rigid connections), and (3) those which exhibit end rotations between simple and rigid connections, referred to as semirigid connections.

To date, the majority of efforts in the development of connection details for composite members has been focused on rigid-type connections.

Design of connections for strength requires knowledge about the capacity of each connection element at the ultimate strength limit state. To ensure satisfactory performance of connections at ultimate strength limit strength, failure of connection elements must be prevented or controlled. The objective in design is to prevent damage to the connection at its ultimate strength limit state and shift the failure locations to other parts of the structure. Connections could finally fail if the applied load level exceeds a certain limit. As a result, it is desirable to proportion the connection so that it will fail in a "controlled" and "desirable" manner. For instance, design of connection elements could be "controlled" through proportioning such that at the strength limit state the connection elements fail by yielding and not weld fracture. Yielding and, finally, fracture of steel elements of connections are usually "desirable" modes of failure in comparison to weld fracture, which could take place without warning.

For most composite connections, another major consideration is the ability to inspect the connection after a major event. For instance, after an earthquake, one should be able to inspect the connection and make judgments as to the safety of the connection. Unfortunately, most elements of composite connections are not easily accessible and their full inspection is not feasible. Therefore, the designer needs to proportion the connection elements such that the failure of "hidden" elements is prevented.

10.2.2 Stability

Connection elements could fail as a result of buckling (elastic or inelastic) of connection elements. This mode of failure, however, is not usually a major concern in connection design.

10.2.3 Serviceability

Connections, as with any other member of the structure, should perform satisfactorily at different limit states. At service load levels, performance of connections should not adversely affect the behavior of the structure. For instance, at service load levels connections could be subjected to a large number of load cycles. These loads could be generated by wind loads or machinery in the case of industrial buildings. Although these loads could be substantially lower than the ultimate load-carrying capacity of each connection element, the connection could develop fatigue cracking, which could result in failure.

Large flexibility at the connection level could result in large interstory drift and member deflections. Therefore, the selection of the connection types at various floor levels could be dictated by the service limit state.

10.2.4 Cyclic behavior

Connections could fail under a large (high-cycle fatigue) or small (low-cycle fatigue) number of cyclic loadings. In the case of high-cycle fatigue, the magnitude of the applied stress is relatively low. Cracking in bridge elements is caused by high-cycle fatigue. On the other hand, the level of applied stress in the case of low-cycle fatigue is relatively high and could approach the yield strength of the connected elements. During major earthquakes, connections in buildings could experience a few cycles of loading with relatively high stress levels at each cycle. Failure of connections by low-cycle fatigue is confined to earthquake-type loading. The amount of available information on low-cycle fatigue characteristics of connections is limited. This is especially true for composite connections. Principles of fracture mechanics and fatigue could be used to establish life of connections under variable cyclic loading. Two approaches could be undertaken. Full-scale testing of connections under constant and variable amplitude loading provides the most reliable information. In the absence of such information, designers could identify the high stress points within the connection and possible load histories that that particular point within the connection could experience during an earthquake. Information on cyclic behavior of different materials, obtained from simple tension-type specimens, is available. Knowledge of the cyclic load history for the portion of the connection with the highest stress and available damage models for particular materials could then be used to estimate the life of connections under cyclic loading.

However, it should be noted that predicting the life of connections under cyclic loading is a very complex process and its accuracy, in many cases, depends on the experience and judgment of the designer. One of the major questions is estimating the load history that the connection could experience during an earthquake. In addition, it is necessary to conduct nonlinear dynamic time-history analyses, incorporating connection behavior (through inclusion of moment-rotation characteristics of the connection). Fortunately, in general, connections in major earthquake events are subjected to a very few cycles of loading with high stress levels. In general, bolted connections demonstrate better cyclic behavior than welded connections. Behavior of welded connections depends, to a large extent, on quality control and workmanship.

10.3 Beam-to-Wall Connections

10.3.1 Introductory remarks

Structural walls/cores are commonly used for lateral load strength and stiffness. For low- to moderate-rise buildings, up to 25 to 30 stories, the walls/cores can be used to provide a majority of the lateral force resistance. For taller buildings, the use of dual systems is more common, where the perimeter frames are engaged with the walls/cores. Outrigger beams are framed between the core walls and columns (which may be all steel or composite) in the perimeter frame. Core walls can effectively be formed by coupling individual wall piers, which may be slip-formed to accelerate construction, with the use of reinforced concrete or steel/steel-concrete composite coupling beams. The floor plan of a representative hybrid building is shown in Fig. 10.1. The walls may be

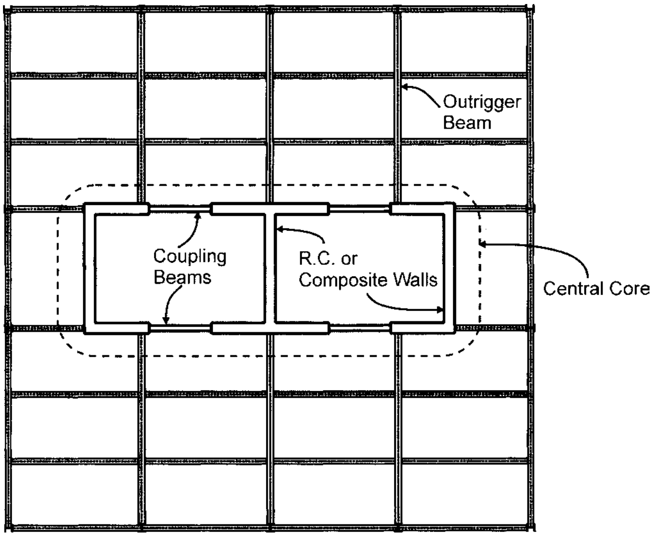


Figure 10.1 Structural components of core wall frame systems.

reinforced conventionally, that is, consisting of longitudinal and transverse reinforcement, or may include embedded structural steel members in addition to conventional reinforcing bars. The successful performance of such hybrid structural systems depends on the adequacy of the primary individual components which are the walls/cores, steel frames, and frame-core connections. The focus of this section is on the connections between outrigger beams and walls and the connections between steel/steel-concrete composite coupling beams and walls. Issues related to design of steel/steel-concrete composite coupling beams and connections between floor diaphragms and walls are also discussed.

10.3.2 Qualitative discussion about outrigger beam-wall connection and coupling beam-wall connection

Connections between walls and steel/composite coupling beams or outriggers depend on whether the wall boundary element is reinforced conventionally or contains embedded structural steel columns, the level of forces to be developed, and whether the walls are slip- formed or cast conventionally. A summary of possible connections is provided in the following.

10.3.2.1 Coupling beam-wall connection. Well-proportioned coupling beams above the second floor are expected to dissipate a majority of the input energy during severe earthquakes. Coupling beams will, therefore, undergo large inelastic end rotations and reversals, and adequate connection between coupling beams and wall piers becomes a critical component of the overall system behavior. The connection varies depending on whether reinforced concrete or steel/steel-concrete composite coupling beams are used. A comprehensive discussion for reinforced concrete coupling beams and their connections to walls is provided elsewhere (for example, Barney et al., 1978; Paulay, 1980, 1986; Aktan and Bertero, 1981; Paulay and Binney, 1975; Paulay and Santhakumar, 1976).

10.3.2.2 Steel/steel-concrete coupling beams. Structural steel coupling beams provide a viable alternative, particularly in cases where height restrictions do not permit the use of deep reinforced concrete beams, or where the required capacities and stiffness cannot be developed economically by a concrete beam. The member may be encased with a varying level of longitudinal and transverse reinforcement.

If the wall boundary elements include embedded structural steel columns, the wall-coupling beam connection is essentially identical to steel beams and columns but with some modifications. For steel boundary columns located farther away than approximately 1.5 to 2 times the beam depth from the edge, the beam forces can be transferred to the core by the bearing mechanism mobilized by the beam flanges, as illustrated in Fig. 10.2. In such cases, the beam-column connection becomes less critical, and the necessary embedment length can be computed based on a number of available methods, as discussed in Sec. 10.3.3.3. If the embedded steel boundary column is located within approximately 1.5 times the beam depth from the wall edge, the forces can be transferred by mobilizing the internal couple involving the column axial load and bearing stresses near the face, as

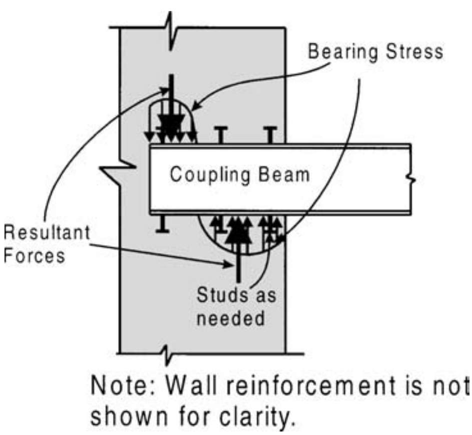


Figure 10.2 Transfer of coupling beam forces through bearing.

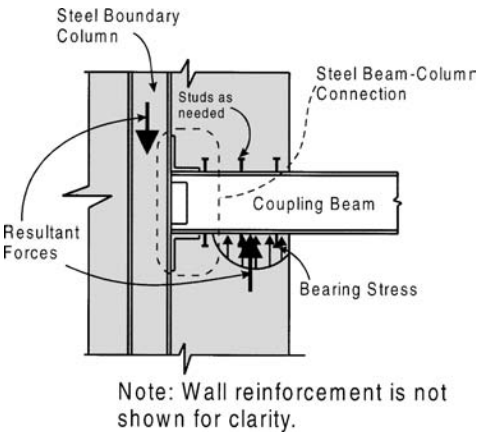


Figure 10.3 Transfer of coupling beam forces through bearing and beam-column connections.

shown in Fig. 10.3. Clearly, the beam-column connection becomes critical in mobilizing this mechanism. The connection between the coupling beam and steel boundary column is expected to be enhanced by the presence of concrete encasement as indicated by a recent study (Leon et al., 1994) which shows improved performance of encased riveted beam column. Due to insufficient data, however, it is recommended to ignore the beneficial effects of the surrounding concrete, and to follow standard design methods for steel beam-column connections. Outrigger beams may also be directly attached to columns which are closer to the core face and protruded beyond the column. This detail is illustrated in Fig. 10.4. Considering the magnitude of typical coupling beam forces, the steel boundary column may deform excessively, particularly if the column is intended to serve as an erection column, leading to splitting of the surrounding concrete and loss of stiffness. Adequate confinement around the column and headed

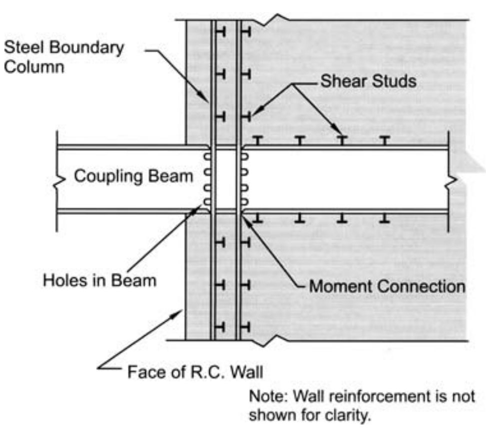


Figure 10.4 Transfer of coupling beam forces through a direct beam-column connection.

studs improves the behavior by preventing separation between the steel column and surrounding concrete.

If the wall boundary element is reinforced with longitudinal and transverse reinforcing bars, a typical connection involves embedding the coupling beam into the wall and interfacing it with the boundary element, as illustrated in Fig. 10.5. The coupling beam has to be embedded adequately inside the wall such that its capacity can be developed. A number of methods may be used to calculate the necessary embedment length (Marcakis and Mitchell, 1980; Mattock and Gaafar, 1982). These methods are variations of Precast Concrete Institute guidelines (PCI) for design of structural steel brackets embedded in precast reinforced concrete columns. Additional details regarding the design methodology are provided in Sec. 10.3.3.3. A second alternative is possible, particularly when core walls are slip-formed. Pockets are left open in the core to later receive coupling beams. After the forms move beyond the pockets at a floor, steel beams are placed inside the pockets and grouted. This detail is illustrated schematically in Fig. 10.6. Calculation of the embedment length is similar to that used for the detail shown in Fig. 10.5.

10.3.2.3 Outrigger beam-wall connection. In low-rise buildings, up to 30 stories, the core is the primary lateral load–resisting system, the perimeter frame is designed for gravity loads, and the connection

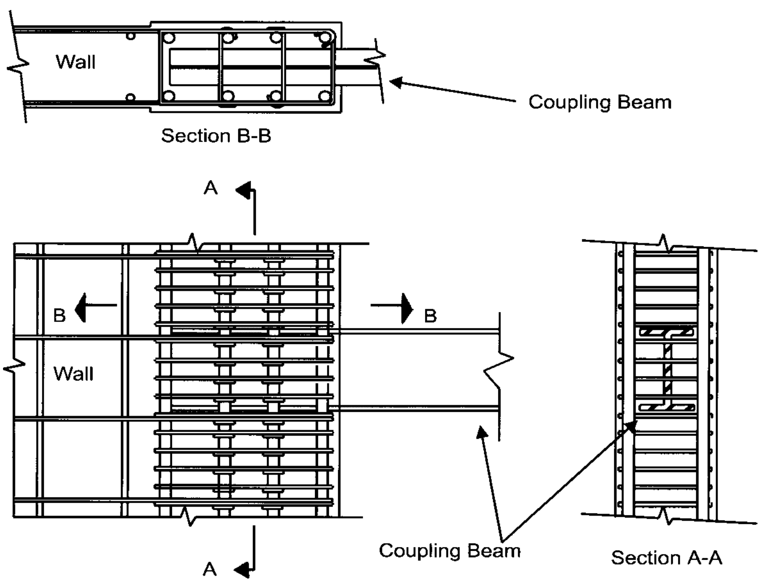


Figure 10.5 Coupling beam-wall connection for conventionally reinforced walls.

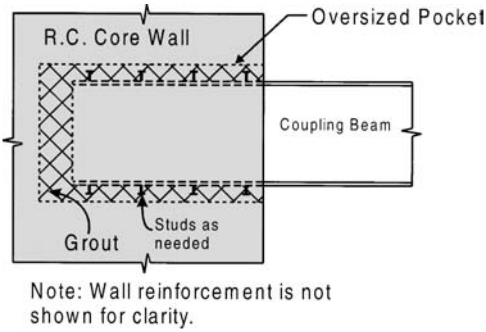


Figure 10.6 A possible coupling beam-wall connection for slip-formed walls.

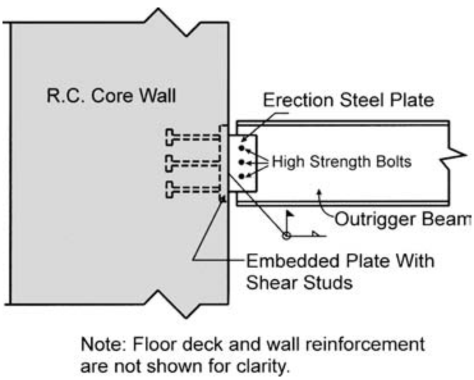


Figure 10.7 Shear connection between outrigger beams and walls.

between outrigger beams and cores is generally a *shear connection*. A typical shear connection is shown in Fig. 10.7. Here, a steel plate with shear studs is embedded in the wall/core during casting, which may involve slip-forming. After casting beyond the plate, the web of the steel beam is welded to the stem of a steel plate (*shear tab*) which is already welded to the plate. Variations of this detail are common.

In taller buildings, moment connections are needed to engage the perimeter columns as a means of reducing lateral deformation of the structural system. For short-span outrigger beams, a sufficient level of stiffness can be achieved by a single structural member (either a built-up or a rolled section). In such cases, a number of different moment-resisting connection details are possible. The detail shown in Fig. 10.8 is suitable for developing small moments (clearly not the full moment capacity of the beam) as found by Roeder and Hawkins (1981) and Hawkins et al. (1980). A larger moment can be resisted by embedding the outrigger beam in the wall during construction, similar to that shown in Fig. 10.5 or 10.6, or by using the detail shown in Fig. 10.9. In the latter option, the outrigger beam is welded to a plate which is anchored in the wall by an embedded structural element similar to the outrigger beam. The latter detail is suitable for slip-formed core walls, as well as for

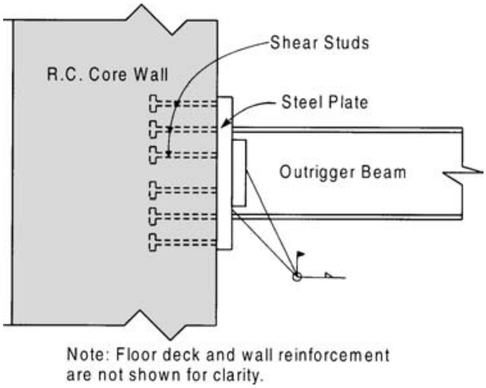


Figure 10.8 Moment connection between outrigger beams and walls (small moments).

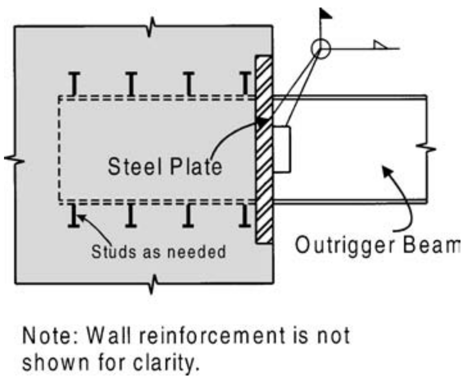


Figure 10.9 Moment connection between outrigger beams and walls (large moments).

conventional construction methods. These details rely on developing an internal couple due to bearing of the beam flanges against the surrounding concrete or grout. If the wall boundary is reinforced with a structural column, the outrigger beam can be directly attached to the wall, as shown in Fig. 10.3 or 10.4.

The span of most outrigger beams is such that a single girder does not provide adequate stiffness, and other systems are needed. Story-deep trusses are a viable choice. As shown schematically in Fig. 10.10, the connection between the top and bottom chords is essentially similar to that used for shear connections between outrigger beams and wall piers.

10.3.2.4 Floor-wall connection. A common component for either of the connections discussed previously is the connection between the floor and walls. In hybrid structures, the floor system consists of a composite metal deck. When the metal deck corrugations are parallel to the core, continuous bent closure plates are placed to prevent slippage of concrete during pouring. These plates may consist of continuous angles, as shown in Fig. 10.11, which are either attached to weld plates already cast in

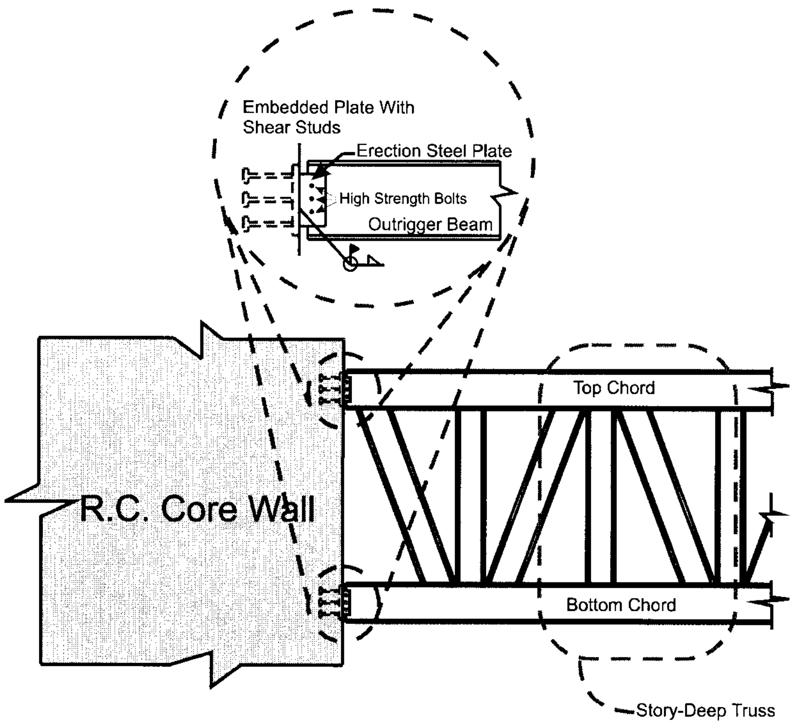


Figure 10.10 Connection between story-deep trusses and walls.

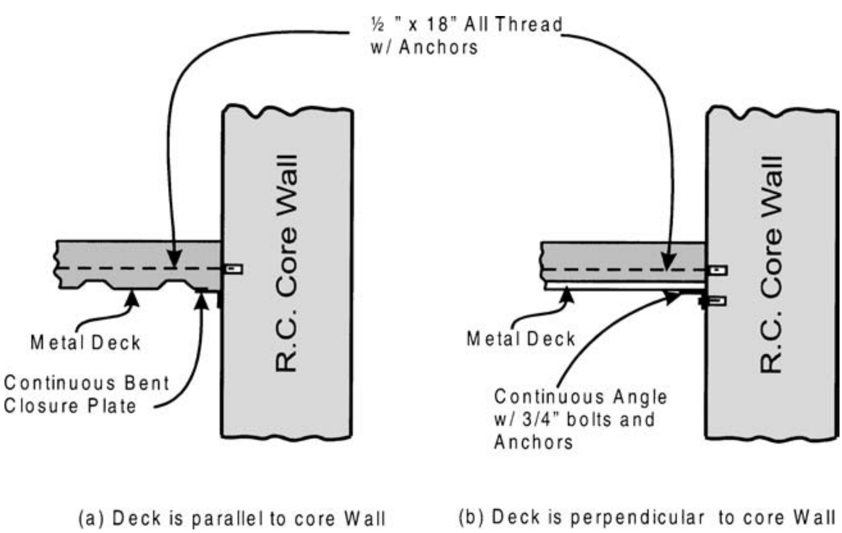


Figure 10.11 Representative connection between composite floors and core walls.

the wall, or anchored directly to the wall. When the metal deck corrugations are perpendicular to the core, the deck is supported by steel angles which are attached to the core typically at 12 to 24 in on center (Fig. 10.11*b*). In addition, dowels at regular intervals (18 in on center is common) are used to transfer lateral loads into the core. Note that for encased coupling beams (that is, steel-concrete composite members), the floor system is a reinforced concrete slab or posttensioned system. For these cases, the floor-wall connection is similar to reinforced concrete slab or posttensioned floor-wall connections.

10.3.3 Design of steel or steel-concrete composite coupling beam-wall connections

10.3.3.1 Analysis. Accurate modeling of coupled wall systems is a critical step, particularly when steel or steel-concrete composite coupling beams are used. Previous studies (Shahrooz et al., 1992, 1993; Gong and Shahrooz, 1998) suggest that steel or steel-concrete composite coupling beams are not fixed at the face of the wall. As part of design calculations, the additional flexibility needs to be taken into account to ensure that wall forces and lateral deflections are computed reasonably well. Based on experimental data (Shahrooz et al., 1992, 1993; Gong and Shahrooz, 1998), the *effective fixed point* of steel or steel-concrete composite coupling beams may be taken as one-third of the embedment length from the face of the wall. The corresponding design model is illustrated in Fig. 10.12.

Stiffness of coupling beams needs to be estimated properly as the design forces and hence detailing of coupling beam-wall connection are impacted. For steel coupling beams, standard methods are used to calculate the stiffness. The stiffness of steel-concrete composite coupling beams needs to account for the increased stiffness due to encasement. Stiffness based on gross transformed section should be used to calculate the upper-bound values of demands in the walls, most notably wall axial force. Cracked transformed section moment of inertia may be used when deflection limits are checked and to compute the maximum wall overturning moment. Note that a previous study suggests that the additional stiffness due to floor slab is lost shortly after composite coupling beams undergo small deformations (Gong and Shahrooz, 1998). Until additional experimental data become available, it is recommended to include the participation of the floor slab for calculating wall axial force. Effective flange width for T beams, as specified in American Concrete Institute (ACI 318), may be used for this purpose. The participation of floor slab toward the stiffness of steel-concrete composite coupling beams may be ignored when drift limits are checked or when the maximum wall overturning moments are computed.

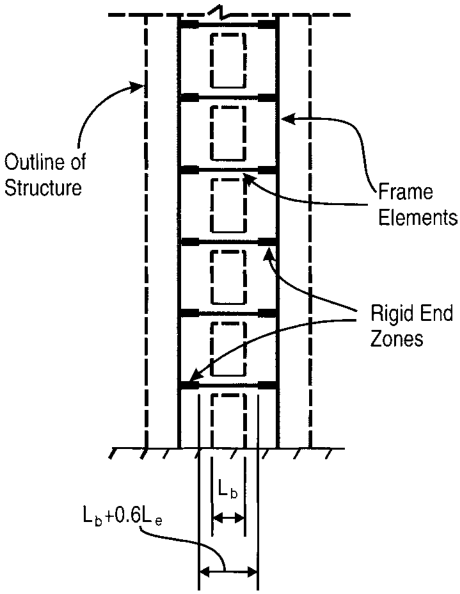


Figure 10.12 Design model for coupled wall systems using steel or composite coupling beams.

10.3.3.2 Design of coupling beam

Steel coupling beams. Well-established guidelines for shear links in eccentrically braced frames (AISC, 2005) may be used to design and detail steel coupling beams. The level of coupling beam rotation angle plays an important role in the number and spacing of stiffener plates which may have to be used. This angle is computed with reference to the collapse mechanism shown in Fig. 10.13 which corresponds to the expected behavior of coupled wall systems, that is, plastic hinges form at the base of walls and at the ends of coupling beams. The value of θ_p is taken as $0.4R\theta_e$ in which elastic interstory drift angle θ_e is computed under code level lateral loads (for example, NEHRP, 1994; UBC, 1994). The minimum value of the term $0.4R$ is 1.0. Knowing the value of θ_p, shear angle γ_p is calculated from Eq. (10.1):

$$\gamma_p = \frac{\gamma_p L}{L_b + 0.6\,L_e} \tag{10.1}$$

Note that in this equation, the additional flexibility of steel/composite coupling beams is taken into account by increasing the length of the coupling beam to $L_b + 0.6L_e$. This method is identical to that used for calculating the expected shear angle in shear links of eccentrically braced frames with the exception of the selected collapse mechanism.

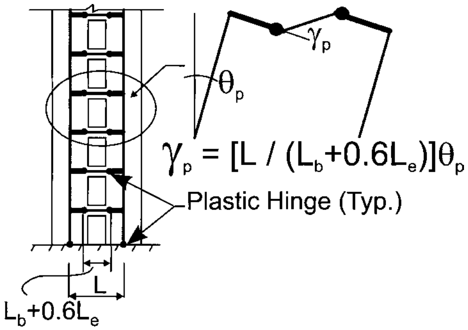


Figure 10.13 Model for calculating shear angle of steel or composite coupling beams.

Steel-concrete composite coupling beams. Previous research on steel-concrete composite coupling beams (Shahrooz et al., 1992, 1993; Gong and Shahrooz, 1998) indicates that nominal encasement around steel beams provides adequate resistance against flange and web buckling. Therefore, composite coupling beams may be detailed without web stiffener plates. Due to inadequate data regarding the influence of encasement on local buckling, minimum flange and web thicknesses similar to steel coupling beams need to be used.

10.3.3.3 Connection design. The connection becomes more critical when steel or steel-concrete composite coupling beams are used. For the details shown in Fig. 10.3 or 10.4, standard design methods for steel beam-column connections can be followed. If the connection involves embedding the coupling beam inside the wall (see Fig. 10.5 or 10.6), the required embedment length is calculated based on mobilizing the moment arm between bearing forces C_f and C_b, as shown in Fig. 10.14. This model was originally proposed by Mattock and Gaafar (1982) for steel brackets embedded in precast concrete columns. Previous studies

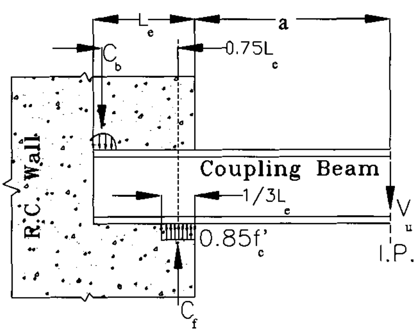


Figure 10.14 Model for computing embedment length.

(Shahrooz et al., 1992, 1993; Gong and Shahrooz, 1998) have shown the adequacy of this model for steel or steel-concrete composite coupling beams. This model calculates the required embedment length, L_e, from Eq. (10.2):

$$V_u = 48.6\sqrt{f'_c}\left(\frac{t_{\text{wall}}}{b_f}\right)^{0.66} \beta_1 b_f L_e \left(\frac{0.58 - 0.22\beta_1}{0.88 + (a/L_e)}\right) \tag{10.2}$$

For the detail shown in Fig. 10.6, the value of f'_c in Eq. (10.2) is to be taken as the minimum of the compressive strength of the wall and grout.

The value of V_u in Eq. (10.2) should be selected to ensure that the connection does not fail prior to fully developing the capacity of the coupling beam. For steel coupling beams, V_u is taken as the plastic shear capacity of the steel member as computed from Eq. (10.3):

$$V_p = 0.6\, F_y\, (h - 2t_f)\, t_w \tag{10.3}$$

To account for strain hardening, it is recommended that F_y be taken as 1.25 times the nominal yield strength.

The contribution of encasement toward shear capacity of composite coupling beams needs to be taken into account when the embedment length is calculated. Embedment length should be adequate such that most of the input energy is dissipated through formation of plastic hinges in the beam and not in the connection region (Shahrooz et al., 1992, 1993; Gong and Shahrooz, 1998). In lieu of fiber analysis, shear capacity of composite coupling beams V_u can be computed from Eq. (10.4), which has been calibrated based on a relatively large number of case studies (Gong and Shahrooz, 1998):

$$\begin{aligned} V_u &= 1.56\,(V_{\text{steel}} + V_{\text{RC}}) \\ V_{\text{steel}} &= 0.6\, F_y\, t_w\, (h - 2t_f) \\ V_{\text{RC}} &= 2\sqrt{f'_c}\, b_w\, d + \frac{A_v f_y d}{s} \end{aligned} \tag{10.4}$$

In this equation, the nominal values of F_y and f'_c (in psi) are to be used because the equation has been calibrated to account for strain hardening and material overstrength.

Additional bars attached to the beam flanges (*transfer bars*) can contribute toward load resistance. These bars can be attached through mechanical half couplers which have been welded onto the flanges. The embedment length as computed by Eq. (10.2) can be modified to account for the additional strength (Gong and Shahrooz, 1998). However, to

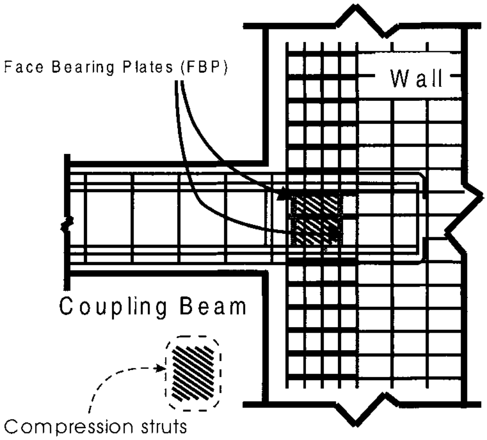


Figure 10.15 Face-bearing plates.

ensure that the calculated embedment length is sufficiently large to avoid excessive inelastic damage in the connection region, it is recommended that the contribution of transfer bars be neglected.

A pair of stiffener plates (on both sides of the web) placed along the embedment length will mobilize compression struts in the connection region as depicted schematically in Fig. 10.15. These stiffener plates are commonly referred to as *face-bearing plates.* The first face-bearing plate should be inside the confined core of the wall boundary element. The distance between the face-bearing plates should be such that the angle of compression struts is approximately 45° (hence, the distance between the plates should be about equal to the clear distance between the flanges). To ensure adequate contribution of the face-bearing plates, the width of each face-bearing plate should be equal to the flange width on either side of the web. The thickness of the face-bearing plates can be established based on available guidelines for the detailing of shear links in eccentrically braced frames (AISC, 2005).

10.3.3.4 Design example. An example is used to illustrate the procedure for computing the required embedment length of steel or steel-concrete composite coupling beams. A representative connection at floor 7 of the structure shown in Fig. 10.16 is designed in this example. The building in this example has 20 floors. The coupling beams are encased, that is, composite, and the walls are assumed to be reinforced only with longitudinal and transverse reinforcement. The clear span of the coupling beam is 8 ft. The thickness of the wall boundary element, t_{wall}, is 22 in. The material properties are. f'_c (for the encasement as well as the wall) = 4 ksi, F_y (yield strength of the web of the steel coupling beam) = 40 ksi, and f_y (yield strength of reinforcing bars in the encasement) = 60 ksi. The cross section of the coupling beam is shown in Fig. 10.17. The effective

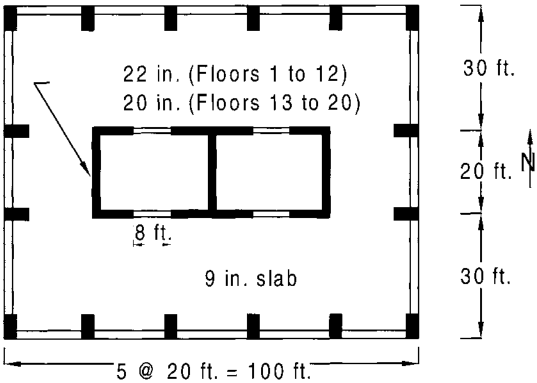


Figure 10.16 Floor plan of example structure.

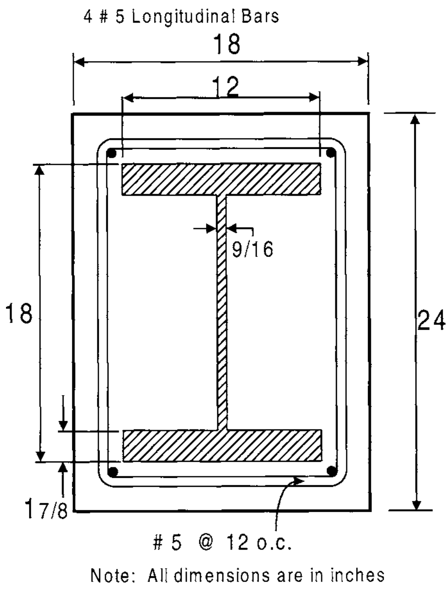


Figure 10.17 Cross section of composite coupling beam.

depth for the concrete element is taken as 21.5 in. The goal is to compute the required embedment length of the steel coupling beam inside the reinforced concrete wall.

The embedment length needs to develop $V_u = 1.56\,(V_{\text{steel}} + V_{\text{RC}})$. The values of V_{RC} and V_{steel} are computed in Eq. (10.5):

$$V_{\text{RC}} = 2\sqrt{f'_c}\,b_w\,d + \frac{A_s f_y d}{s}$$

$$V_{RC} = 2\frac{\sqrt{4000}}{1000}(18)(21.5) + \frac{2(0.31)(60)(21.5)}{12}$$

$$V_{RC} = 116 \text{ kips}$$

$$V_{steel} = 0.6F_y t_w(d - 2t_f)$$

$$V_{steel} = 0.6(40)(9/16)[18 - (2)(1.875)]$$

$$V_{steel} = 193 \text{ kips} \tag{10.5}$$

The embedment length is designed to develop $V_u = 1.56(116 + 193) =$ 480 kips.

$$V_u = 48.6\sqrt{f'_c}\left(\frac{t_{wall}}{b_f}\right)^{0.66} \beta_1 b_f L_e \left(\frac{0.58 - 0.22\beta_1}{0.88 + (a/L_e)}\right) \tag{10.6}$$

Therefore,

$$480 = 48.6\frac{\sqrt{4000}}{1000}\left(\frac{22}{12}\right)^{0.66}(0.85)(12)\,L_e\left(\frac{0.58 - 0.22(0.85)}{0.88 + (48/L_e)}\right) \tag{10.7}$$

By solving Eq. (10.7), the required embedment length is 48.6 in, say 49 in. The final detail is shown in Fig. 10.18. Note that 1.5-in transfer bars have been added to the top and bottom flanges as shown.

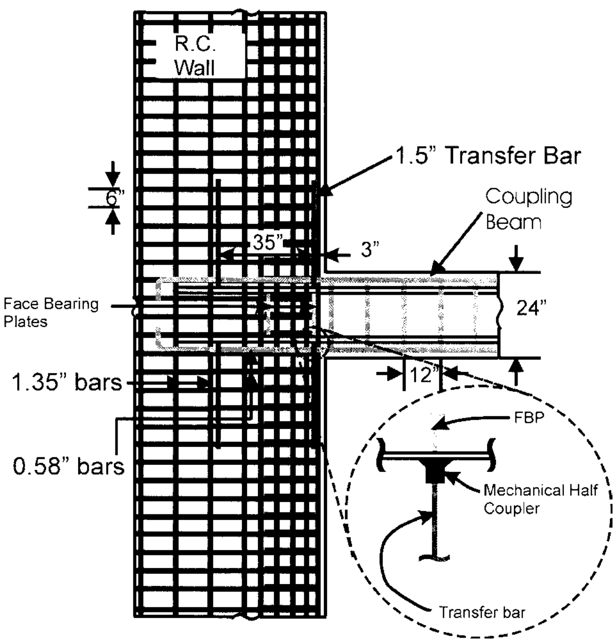


Figure 10.18 Connection detail.

10.3.4 Design of outrigger beam-wall connections

10.3.4.1 Shear connections. As explained in Sec. 10.3.2.3, outrigger beams are typically connected to core walls through *shear connections* similar to that shown in Fig. 10.7. Although this connection provides some moment resistance, it is generally accepted that the connection is flexible and does not develop large moments. The main design issues are (1) the connection between the steel outrigger beam and shear tab which is welded onto the embedded plate and (2) the transfer of forces, which are gravity shear force and diaphragm force, as shown in Fig. 10.19, to the wall. Note that the diaphragm force may be tensile or compressive, and the line of action of gravity shear is assumed to be along the bolts according to standard practice. The outrigger beam-shear tab connection is a typical shear connection, and common design methods for steel structures (AISC, 2005) are followed for this purpose. The most critical part is the transfer of forces, particularly tensile diaphragm forces, from the shear tab to the core wall, which is achieved by headed studs. To ensure adequate safety against stud failure, the following design methodology is recommended. This method is based on a research conducted by Wang (1979):

1. Based on an assumed layout of studs, establish the tensile capacity as the lesser of the strength of the stud or concrete cone. Available guidelines (from PCI) can be used for this purpose.

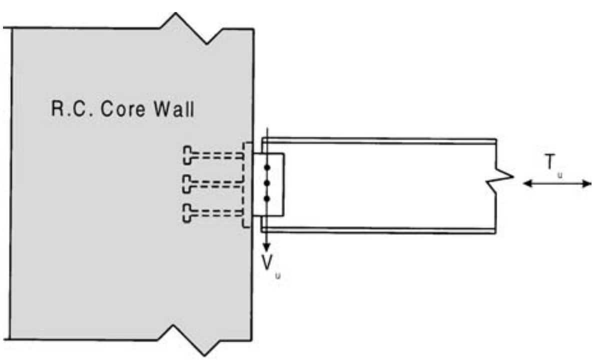


Figure 10.19 Forces on shear connection between outrigger/collector beams and core walls.

2. Assuming that all the applied shear is resisted by the studs in the compression region, calculate the required number of studs. The shear capacity is taken as the smaller of (*a*) shear capacity of a single stud, which can be calculated based on available guidelines (PCI) and (*b*) tensile capacity calculated in step (1). Use the same number of studs in the tension zone. Once the required number of studs is known, compute shear strength governed by concrete failure.
3. Using the stud arrangement obtained in step (2), compute tensile capacity of the stud group.
4. Increase the value of T_u by 50% to ensure adequate ductility.
5. Based on the model shown in Fig. 10.20 and the formulation shown in Eq. (10.8), calculate the depth of compression region, k_d:

$$k_d = \frac{T_{\text{capacity}} - 1.5T_u}{(0.85f'_c)b} \tag{10.8}$$

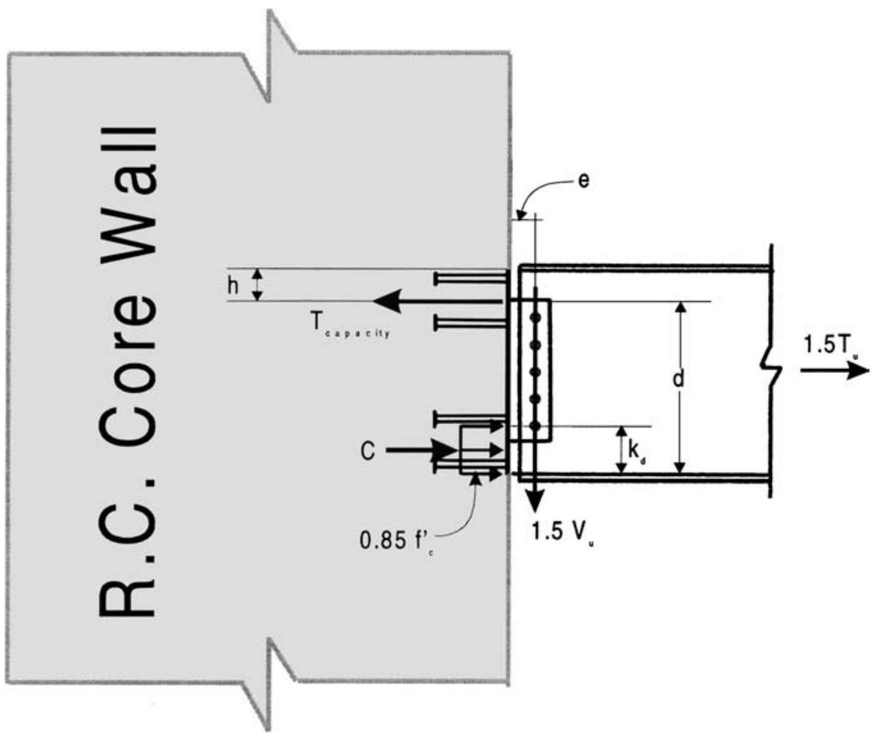


Figure 10.20 Design model for design of outrigger beam-wall shear connections.

6. Calculate the required depth of the embedded plate from Eq. (10.9):

$$d = \frac{1.5eV_u + 0.425bf'_c k_d^2 + 0.75T_u h}{0.85\, bf'_c k_d + 0.75T_u} \tag{10.9}$$

$$\text{depth} = d + h$$

   Note that in this equation, the value of gravity shear V_u is amplified by 1.5 to ensure a ductile mode of failure.

7. Check the capacity of studs under combined actions of tension and shear. For this purpose, the shear may be assumed to be resisted equally by the tension and compression studs, but the tensile force is resisted by tension studs. Available interaction equations in PCI guidelines can be used for this purpose.

10.3.4.2 Moment connections. As mentioned previously in Sec. 10.3.2.3, outrigger beams may be attached to core walls through moment connections to enhance the overall structural stiffness. The basic force-transfer mechanism for the connections shown in Fig. 10.2, 10.5, 10.6, or 10.9 is similar to that discussed for coupling beams embedded inside core walls. For the connection shown in Fig. 10.8, the aforementioned design procedure for shear connections can generally be followed, but the term, $1.5eV_u$, in Eq. (10.9) is replaced by $1.5M_u$. Once again, the calculated design moment, M_u, has been increased by 50% to ensure a ductile behavior. The connections for top and bottom chords of story-deep outrigger trusses (Fig. 10.10) are similar to shear connections, and are designed according to the formulation described in Sec. 10.3.4.1.

10.3.4.3 Floor-wall connections. In a structure with the floor plan shown in Fig. 10.1, it is possible to transfer diaphragm forces directly to core walls through the outrigger beams, which also serve as collector elements. In such cases, the connection between composite floor systems and core walls, which were discussed in Sec. 10.3.2.4, has to simply resist the gravity shear. The connection between the necessary supporting elements and core walls is designed according to established guidelines (from PCI). To reduce the demands on outrigger beam-wall connections, the floor system may be designed to participate in the transfer of diaphragm forces to the core walls. Dowels at regular spacing can be used for this purpose. The dowels have to be embedded adequately in the floor slab, and be anchored to the wall so that their capacity can be developed. These dowels

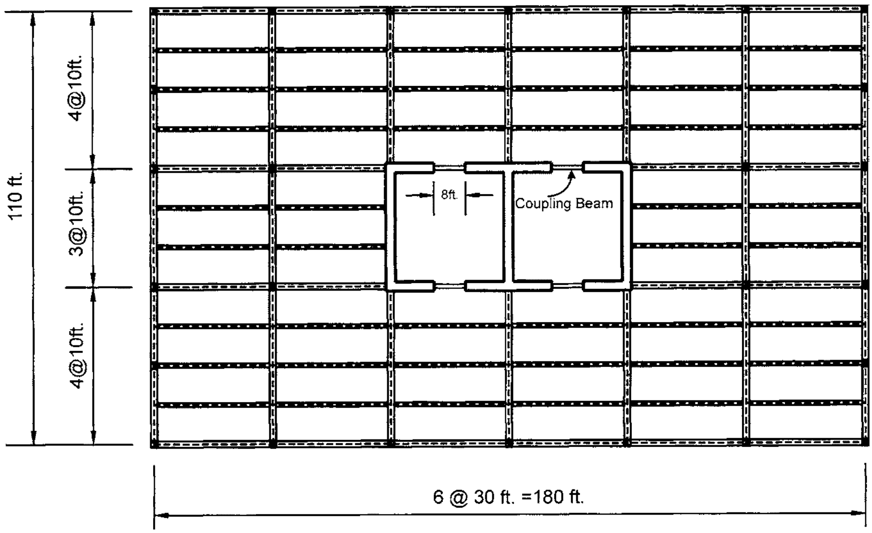


Figure 10.21 Plan view of design example.

have to resist the portion of tensile diaphragm force not resisted by the collector element.

10.3.4.4 Design example for shear connections. An example of shear connections between outrigger beams and core walls is illustrated in this section. The example is with reference to a 15-story building with the floor plan shown in Fig. 10.21. The calculated forces for the outrigger beam in floor 5 are $T_u = 40$ kips and $V_u = 93$ kips. The outrigger beam is W 24 × 55, and the core walls are 18 in thick. The concrete compressive strength of the wall is 6000 psi.

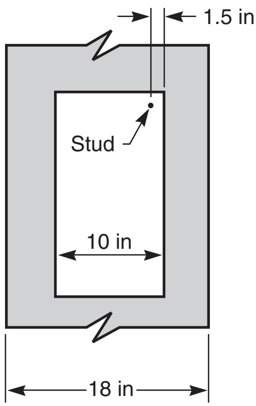
- *Design of shear tab:* The shear tab is designed and detailed by following standard design practice for steel structures (AISC, 2005). The shear tab dimensions are 15.5 in deep × 4.5 in wide × ½ in thick. The shear tab is welded to the embedded steel plate through ¼-in fillet weld. Five 1-in A490 bolts are used to connect the outrigger beam to the shear tab.
- *Design of embedded steel plate:* Try ¾-in-diam studs with 7 in of embedment:

  1. *Tensile capacity of studs:*

$$\phi P_s = \phi 0.9 A_b f_y = (1)(0.9)(0.4418)(60) = 23.85 \text{ kips}$$

Assuming that the stud is located as shown below, the tensile strength governed by concrete failure is

$$\phi P_c = 10.7 l_e (l_e + d_h)\sqrt{f'_c}\,\frac{d_e}{l_e}$$



in which $l_e = 7\tfrac{3}{8} = 6.625$ in
$d_e = 5.5$ in
$\phi P_c = 10.7(6.625)(6.625 + 1.25)\sqrt{6000}\,(5.5/6.625)$
$\phi P_c = 35{,}900$ lb $= 35.9$ kips

Therefore, use ϕP_s.

2. *Shear strength:*

$$\phi V_s = \phi 0.75 A_b f_y = (1)(0.75)(0.4418)(60) = 19.9 \text{ kips}$$

The shear strength is the smaller of ϕV_s and tensile strength. Hence, shear strength = 19.9 kips. The number of required studs = 93/19.9 = 4.7, say 5 studs. Compute shear strength governed by concrete failure. Since the edge distance > $15d_b$ (= 11.25 in),

$$\phi V_c = \phi 800\, A_b \sqrt{f'_c}\, n$$

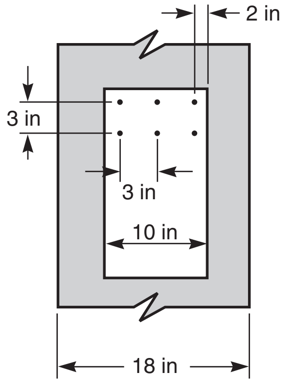
Therefore, $\phi V_c = 0.85(800)(0.4418)(\sqrt{6000})5 = 116354$ lb = 116.4 kips, which is larger than V_u, ok. To have an even number, use six studs in both tension and compression zones.

3. *Tensile strength of stud groups:* Assuming the stud pattern shown below, the capacity is computed from the following equation:

$$\phi P_c = \phi(4)\sqrt{f'_c}(x + d_{e1} + d_{e2})(y + 21_e)$$

$$\phi P_c = 0.85(4)(\sqrt{6000})(6 + 6 + 6)[3 + 2(6.625)] = 77{,}033 \text{ lb}$$

$$\phi P_c = 77.0 \text{ kips} > 1.5T_u, \text{ ok}$$



4. *Size the embedded plate:* From Eq. (10.8),

$$k_d = \frac{77.0 - 1.5(40)}{0.85(6)(10)} = 0.334 \text{ in}$$

Assuming that the plate extends 1 in above the top stud, the value of h in Eq. (10.9) is 2.5 in. As seen from Fig. 10.22, the value of $e = 2.75$ in. Use Eq. (10.9) to solve for d:

$$d = \frac{1.5(93)(2.75) + 0.425(6)(0.334)^2(10) + 0.75(40)(2.5)}{0.75(40) + 0.85(6)(0.334)(10)} = 9.76 \text{ in}$$

Therefore, the depth of the embedded plate is $d + h = 2.5 + 9.76 = 12.3$ in, say 12.5 in. Note that this depth is less than that required for the shear tab. Assuming that the embedded plate extends $\frac{3}{4}$ in

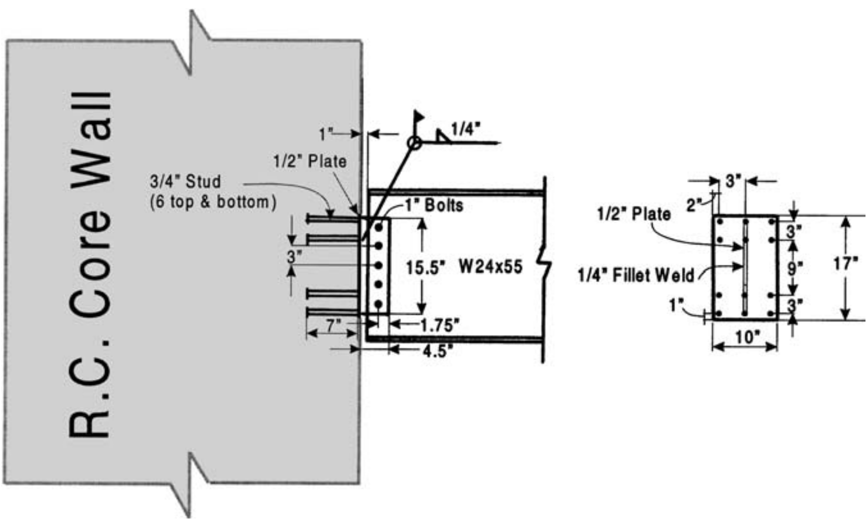


Figure 10.22 Detail of shear connection between outrigger/collector beam and core wall.

beyond the shear tab, the required depth of the embedded plate is 0.75 + 15.5 + 0.75 = 17 in.

According to PCI guidelines, the plate thickness is taken as two-thirds of the diameter of the stud. Hence, the plate thickness is 0.5 in.

The final design is sketched in Fig. 10.22.

5. *Check the studs for combined effects of shear and tension:* Use the following interaction equations recommended by PCI:

$$\frac{1}{\phi}\left[\left(\frac{T}{P_c}\right)^2 + \left(\frac{V_u}{V_c}\right)^2\right] \leq 1.0$$

$$\left[\left(\frac{T}{P_s}\right)^2 + \left(\frac{V_u}{V_s}\right)^2\right] \geq 1.0$$

Using the free-body diagram shown in Fig. 10.23, the value of T can be computed as follows:

$$\Sigma F_x = 0;$$

$$0.85f'_c k_d b + 1.5T_u - T = 0.85(6)(10)\,k_d + 1.5(40) - T = 0$$

$$T = 51\,k_d + 60$$

$$\Sigma M_{\text{about } T} = 0$$

$$1.5T_u\,(14.5 - 8.5) + 0.85f'_c k_d b\left(14.5 - \frac{k_d}{2}\right) - 1.5V_u e = 0$$

$$1.5(40)(6) + 0.85(6)(k_d)10(14.5 - 0.5k_d) - 1.5(93)(2.75) = 0$$

$$25.5k_d^2 - 739.5k_d + 23.63 = 0$$

$$k_d = 0.032 \text{ in}$$

Therefore, $T = 51(0.0322) + 60 = 61.6$ kips

$$P_c = 4\sqrt{f'_c}\,(x + d_{e1} + d_{e2})(y + 21_e)$$
$$= 4\sqrt{6000}(6 + 6 + 6)[3 + 2(6.625)] = 906{,}280 \text{ lb}$$
$$= 90.6 \text{ kips}$$

$$P_s = 0.9(A_s f_y)n = 0.9(0.4418)(60)6$$
$$= 143 \text{ kips (six studs are in tension; } n = 6)$$

$$V_c = 800\,A_b\sqrt{f'_c}\,n = 800(0.4418)\sqrt{6000}(12) = 328{,}530 \text{ lb}$$
$$= 329 \text{ kips}$$

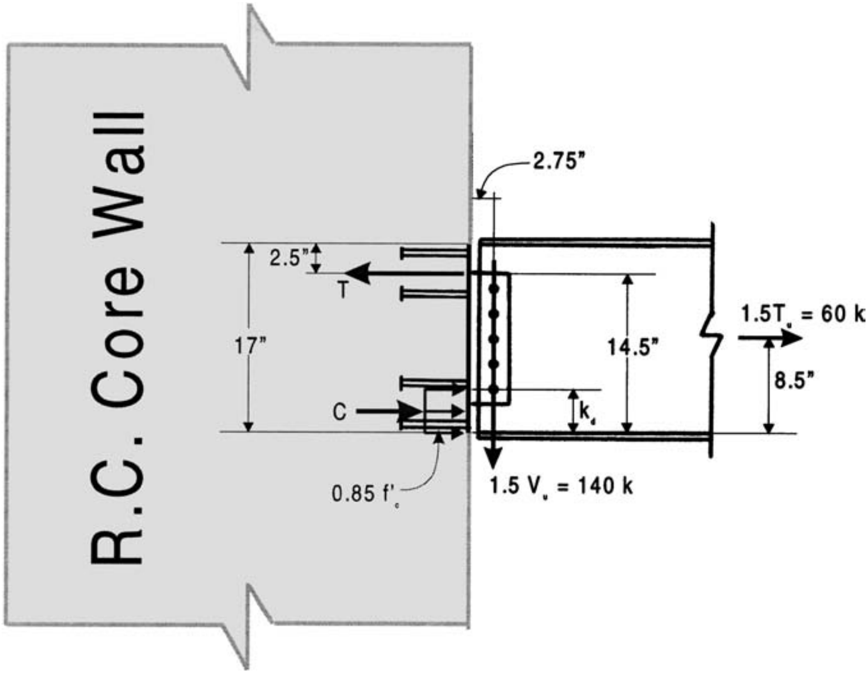


Figure 10.23 Free-body diagram to check final design.

$$V_s = 0.75\, A_b f_y\, n = 0.75(0.4418)(60)(12)$$

$$= 239 \text{ kips (shear is resisted by all studs; } n = 12)$$

Therefore,

$$\frac{1}{0.85}\left[\left(\frac{61.6}{90.6}\right)^2 + \left(\frac{140}{329}\right)^2\right] = 0.76 < 1.0, \text{ ok}$$

$$\left[\left(\frac{61.6}{143}\right)^2 + \left(\frac{140}{239}\right)^2\right] = 0.53 < 1.0, \text{ ok}$$

The final design shown in Fig. 10.22 is adequate.

10.4 Joints between Steel Beams and Reinforced Concrete Columns

10.4.1 Introduction

Composite frames consisting of steel beams and reinforced concrete columns constitute a very cost-effective structural system, especially in tall buildings where the columns have to sustain high axial loads. Concrete columns are known to be more cost-effective than structural steel columns under axial loads. On the other hand, steel beams have the advantages of faster construction and no formwork or shoring required. The combination of concrete columns and steel beams in one system results in the most efficient use of the materials. However, to achieve the full advantage of such system, the beam-column connection must be properly detailed and designed. Due to the current separation of the concrete and steel specifications, the need arises for guidelines to design such connections. The ASCE Task Committee (1994) on Design Criteria for Composite Structures in Steel and Concrete presented guidelines for the design moment resisting joints where the steel beams are continuous through the reinforced concrete column. These guidelines are based on the experimental study by Sheikh et al. (1989) and Deierlein et al. (1989) where 15 two-thirds scale joint specimens were tested under monotonic and cyclic loading. The recommendations were also based on relevant information from existing codes and standards. The following sections summarize the ASCE guidelines. For more information, the reader is referred to the paper by the ASCE Task Committee (1994).

10.4.2 Joint behavior

The joint behavior depends on joint details that activate different internal force transfer mechanisms. Failure of the joint can happen in either one of the two primary failure modes shown in Fig. 10.24. The first mode is the panel shear failure, which results from the transmission of the horizontal flange forces through the joint. Both the steel web and concrete panel contribute to the horizontal shear resistance

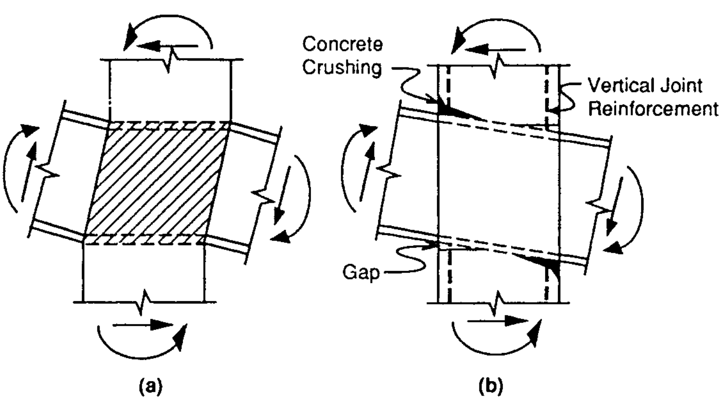


Figure 10.24 Joint failure modes: (*a*) panel shear and (*b*) vertical bearing (*ASCE, 1994*).

in the joint. Attachments that mobilize the concrete panel are discussed in the next section. The second mode is the vertical bearing failure, which results from the high bearing stresses of the compression flange against the column. The joint should be detailed and designed to eliminate the possibility of joint failure and force the failure to occur in the connected members.

10.4.3 Joint detailing

Several configurations of attachments can be used to improve the joint strength (see Fig. 10.25). Details shown in Fig. 10.25*a* and *b* enhance the joint shear capacity through mobilizing a greater portion of the concrete panel. The concrete panel is divided into inner and outer panels. The inner panel is mobilized by the formation of a compression strut through bearing against the FBPs between the beam flanges. Figure 10.26 shows the mobilization of the outer panel by the formation of compression field through bearing against the extended FBPs or steel columns above and below the joint. The FBP may vary in width and may be split for fabrication ease. The ASCE recommendations require that when significant moment is transferred through the beam-column connection, at least FBPs should be provided within the beam depth with the width no less than the flange width. The vertical joint reinforcement shown in Fig. 10.25*c* enhances the joint bearing capacity.

10.4.4 Joint forces

Various forces are transferred to the joint by adjacent members, including bending, shear, and axial loads as shown in Fig. 10.27. Existing data indicate that axial compressive forces in the column can improve the joint strength by delaying the formation of cracks. To simplify the design, and since it is conservative, the axial forces in the column are ignored. Since the axial forces in the beam are generally small, they are also neglected. Accordingly, the design forces are reduced to those shown in Fig. 10.28*a* and *b*. Considering moment equilibrium, the following equation is obtained:

$$\sum M_c = \sum M_b + V_b h - V_c d \tag{10.10}$$

where

$$\sum M_b = (M_{b1} + M_{b2}) \tag{10.11}$$

$$V_b = \frac{V_{b1} + V_{b2}}{2} \tag{10.12}$$

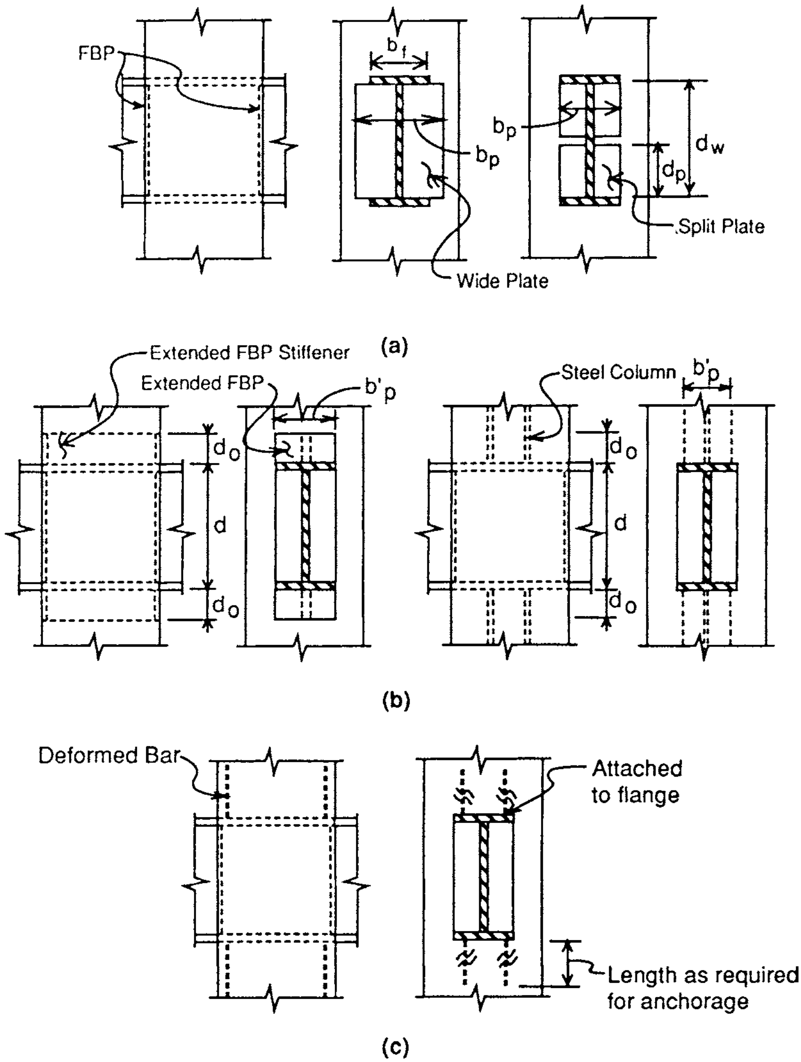


Figure 10.25 Joint details: (*a*) FBP; (*b*) extended FBP and steel column; (*c*) vertical joint reinforcement (*ASCE, 1994*).

$$V_c = \frac{V_{c1} + V_{c2}}{2} \tag{10.13}$$

$$\sum M_c = (M_{c1} + M_{c2}) \tag{10.14}$$

and

$$\Delta V_b = V_{b2} - V_{b1} \tag{10.15}$$

$$\Delta V_c = V_{c2} - V_{c1} \tag{10.16}$$

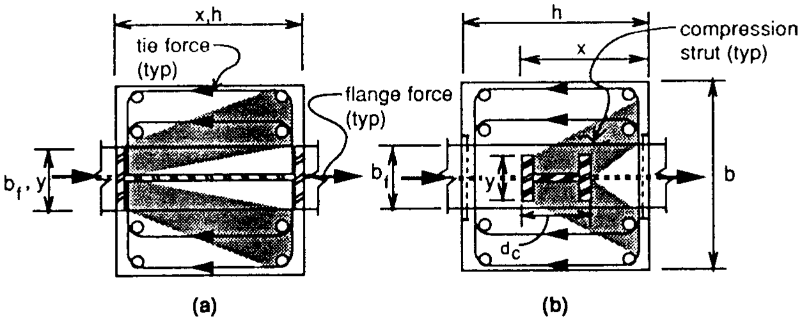


Figure 10.26 Transfer of horizontal force to outer concrete panel: (*a*) extended FBP and (*b*) steel column (*ASCE, 1994*).

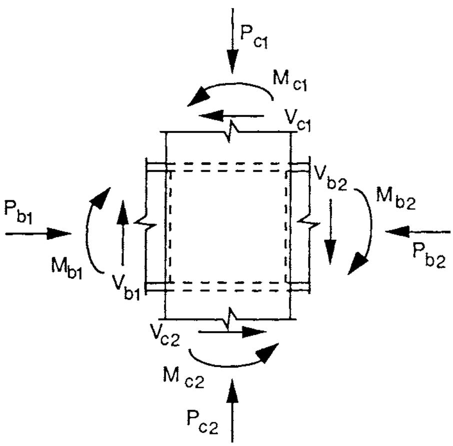


Figure 10.27 Forces acting on joint (*ASCE, 1994*).

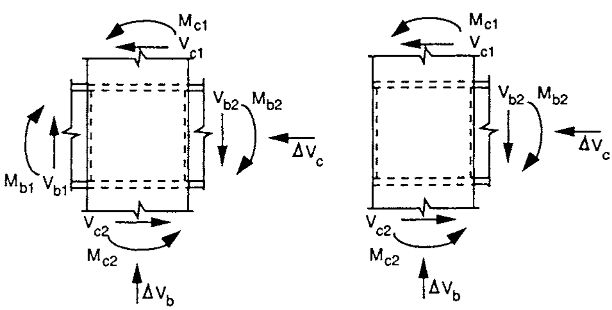


Figure 10.28 Joint design forces: (*a*) interior and (*b*) exterior (*ASCE, 1994*).

10.4.5 Effective joint width

The *effective width of the joint* is defined as the portion of the concrete panel effective in *resisting joint* shear. The concrete panel is divided into inner and outer panels. As shown in Fig. 10.29, the effective joint width, b_j, is equal to the sum of the inner and outer panel widths, b_i and b_o, and can be expressed as

$$b_j = b_i + b_o \tag{10.17}$$

The inner width, b_i, is taken equal to the greater of the FBP width, b_p, or the beam flange width, b_f. Where neither the steel columns nor the extended FBPs are present, the outer panel width, b_o, is taken as zero. Where extended FBPs or steel columns are used, b_o is calculated according to the following:

$$b_o = C(b_m - b_i) < 2d_o \tag{10.18}$$

$$b_m = \frac{b_f + b}{2} < b_f + h < 1.75b_f \tag{10.19}$$

$$C = \frac{x}{h}\,\frac{y}{b_f} \tag{10.20}$$

where b = the concrete column width measured perpendicular to the beam
h = the concrete column depth
y = the greater of the steel column or extended FBP width

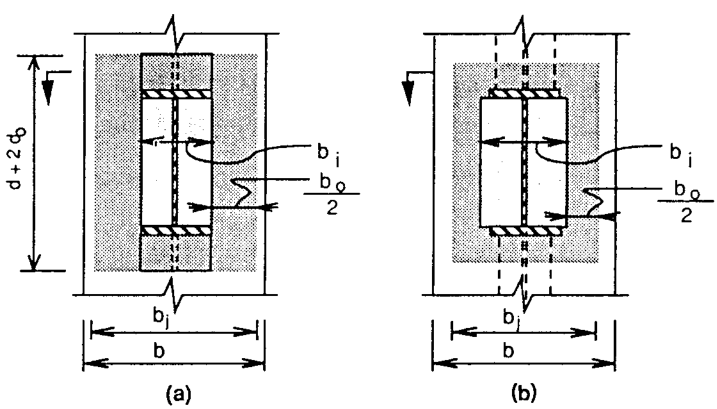


Figure 10.29 Effective joint width (a) extended FBP and (b) wide FBP and column (*ASCE, 1994*).

$x = h$ where extended FBPs are present or $x = h/2 + d_c/2$ when only the steel column is present (see Fig. 10.26)

d_c = steel column width

d_o = additional effective joint depth provided by attachment to the beam and is determined as follows: When a steel column is present, $d_o = 0.25d$ where d = beam depth; when extended FBPs are used, d_o should be taken as the lesser of $0.25d$ or the height of the extended FBPs

10.4.6 Strength requirements

The joint strength is based on the two possible modes of failure mentioned earlier. Joint design strength is obtained by multiplying the nominal strength by a resistance factor, ϕ. Unless otherwise noted, ϕ should be taken equal to 0.70.

10.4.6.1 Vertical bearing. Vertical forces in the joint are resisted by concrete bearing and by joint reinforcement. The equilibrium of the vertical bearing forces is shown in Fig. 10.30, where the moments in the upper and lower columns, M_{c1} and M_{c2}, are replaced with the corresponding forces in the joint reinforcement and the vertical bearing force. To obtain the joint bearing strength, the forces C_c, T_{vr}, and C_{vr} are

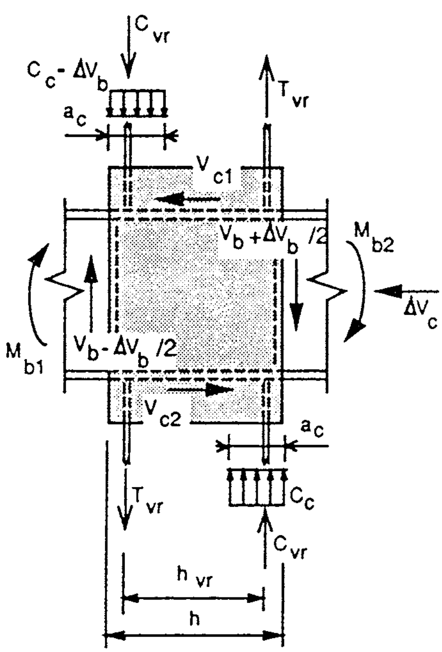


Figure 10.30 Vertical bearing forces (*ASCE, 1994*).

replaced by their nominal values. The bearing strength of the joint is checked according to the following:

$$\sum M_c + 0.35h\Delta V_b \leq \phi[0.7hC_{cn} + h_{vr}(T_{vrn} + C_{vrn})] \qquad (10.21)$$

where $\sum M_c$ = net column moments transferred through the joint
ΔV_b = net vertical beam shear transferred into the column
C_{cn} = the nominal concrete bearing strength
T_{vrn} = the nominal tension strength of the vertical joint reinforcement
C_{vrn} = the nominal compression strength of the vertical joint reinforcement
h_{vr} = the distance between the bars

C_{cn} is calculated using a bearing stress of $2f_c'$ over a bearing area with length $a_c = 0.3h$ and width b_j. The values of $2f_c'$ and $0.3h$ are based on test data. T_{vrn} and C_{vrn} are based on the connection between the reinforcement and steel beam, development of the reinforcement through bond or anchorage to concrete, and the material strength of reinforcement. To avoid overstressing the concrete within the joint, the contribution of the vertical reinforcement is limited by Eq. (10.22):

$$T_{vrn} + C_{vrn} \leq 0.3f_c'b_jh \qquad (10.22)$$

To ensure adequate concrete confinement in bearing regions, three layers of ties should be provided within a distance of $0.4d$ above and below the beam (see Fig. 10.31). The minimum requirement for each layer is given by the following:

$b \leq 500$ mm	four 10-mm bars
$500 \text{ mm} < b \leq 750$ mm	four 12-mm bars
$b > 750$ mm	four 16-mm bars

These ties should be closed rectangular ties to resist tension parallel and perpendicular to the beam.

10.4.6.2 Joint shear. As described in Secs. 10.4.2 and 10.4.3, shear forces in the joint are resisted by the steel web and the inner and outer concrete panels. The three different mechanisms are shown in Fig. 10.32. The horizontal shear strength is considered adequate if the following equation is satisfied:

$$\sum M_c - V_bjh \leq \phi[V_{sn}d_f + 0.75V_{csn}d_w + V_{cfn}(d + d_o)] \qquad (10.23)$$

where V_{sn} = steel panel nominal strength
V_{csn} = the inner concrete compression strut nominal strength
V_{cfn} = the outer concrete compression field nominal strength
V_b = antisymmetric portion of beam shears

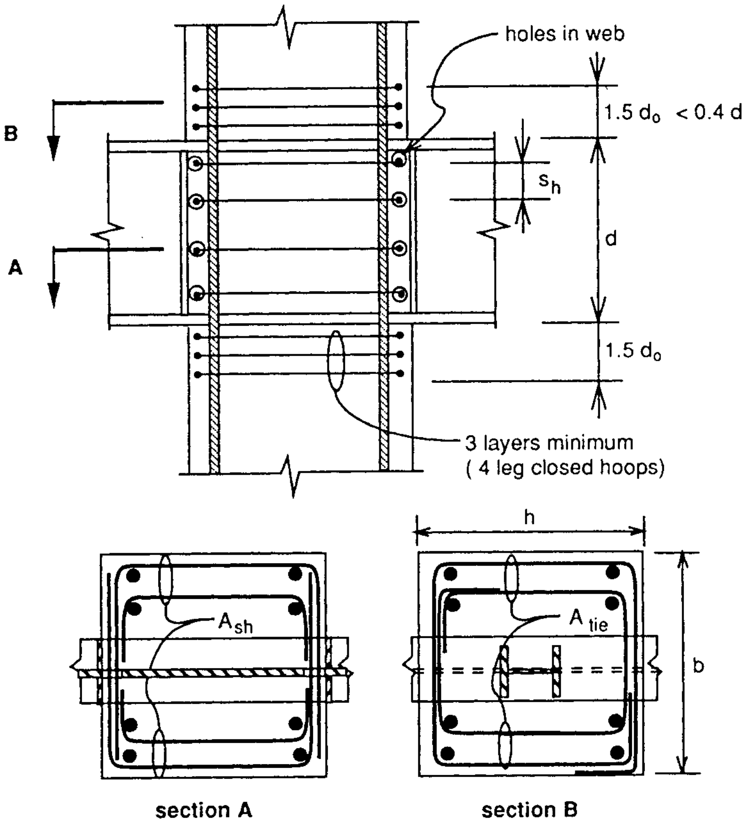


Figure 10.31 Column ties (*ASCE, 1994*).

d_f = the center-to-center distance between the beam flanges
d_w = the depth of the steel web
d_o = additional effective joint depth provided by attachment to the beam
jh = horizontal distance between bearing force resultant and is given by the following:

$$jh = \frac{\sum M_c}{\phi(T_{vrn} + C_{vrn} + C_c) - \Delta V_b/2} \geq 0.7h \tag{10.24}$$

in which

$$C_c = 2f_c'b_ja_c \tag{10.25}$$

$$a_c = \frac{h}{2} - \sqrt{\frac{h^2}{4} - K} \leq 0.3h \tag{10.26}$$

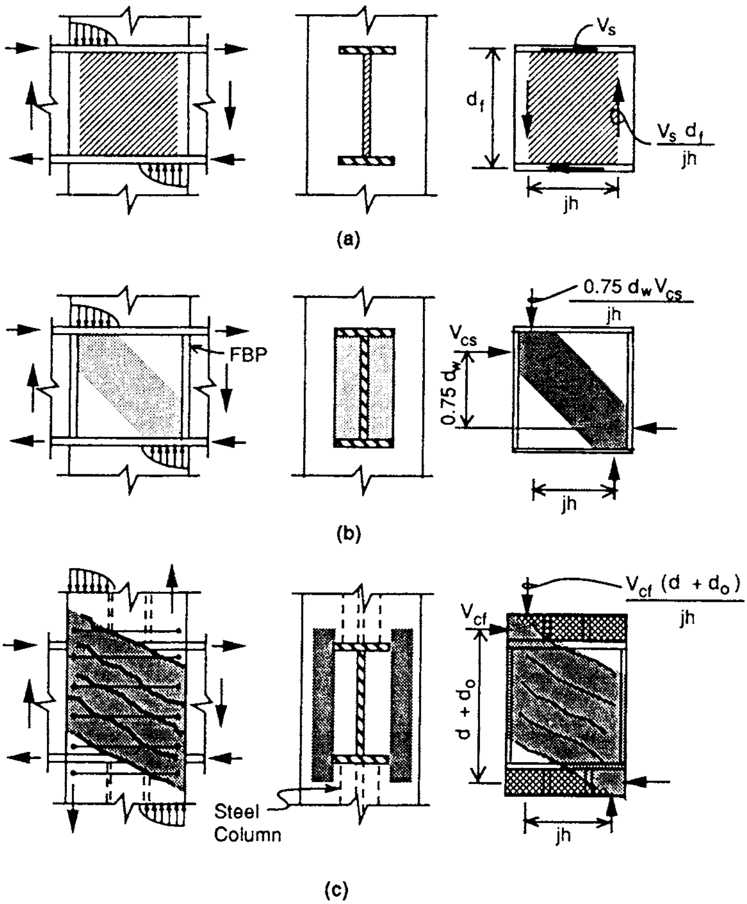


Figure 10.32 Joint shear mechanism (*ASCE, 1994*).

$$K = \frac{1}{\phi 2 f_c' b_j} \left[\sum M_c + \frac{\Delta V_b h}{2} - \phi (T_{vrn} + C_{vrn}) h_{vr} \right] \tag{10.27}$$

In Eq. (10.23), it is assumed that the contributions of the mechanisms are additive. The following sections describe the individual contribution of each of the three different mechanisms.

Steel panel. The steel contribution is given as the capacity of the beam web in pure shear. Assuming the effective panel length to be equal to jh and the average shear yield stress is $0.6F_{ysp}$, the nominal strength of the steel panel, V_{sn}, is expressed as follows:

$$V_{sn} = 0.6 F_{ysp} t_{sp} jh \tag{10.28}$$

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where F_{ysp} = the yield strength of the steel panel and t_{sp} = the thickness of the steel panel.

The vertical shear forces in the steel web cause the beam flanges to bend in the transverse direction. To prevent beam flanges failure, the thickness should satisfy the following:

$$t_f \geq 0.30\sqrt{\frac{b_f t_{sp} F_{ysp}}{h F_{yf}}} \tag{10.29}$$

where F_{yf} is the yield strength of the beam flanges.

Concrete strut. The nominal strength of the concrete compression strut mechanism, V_{csn}, is calculated as follows:

$$V_{csn} = 1.7\sqrt{f_c'} b_p h \leq 0.5 f_c' b_p d_w \tag{10.30}$$

$$b_p \leq b_f + 5t_p \leq 1.5b_f \tag{10.31}$$

where f_c' = the concrete compressive strength, in MPa
b_p = the effective width of FBP, and is limited by Eq. (10.31)
t_p = the thickness of the FBP and should meet the following conditions:

$$t_p \geq \frac{\sqrt{3}}{b_f F_{up}}(V_{cs} - b_f t_w F_{yw}) \tag{10.32}$$

$$t_p \geq \frac{\sqrt{3}V_{cs}}{2b_f F_{up}} \tag{10.33}$$

$$t_p \geq 0.2\sqrt{\frac{V_{cs} b_p}{F_{yp} d_w}} \tag{10.34}$$

$$t_p \geq \frac{b_p}{22} \tag{10.35}$$

$$t_p \geq \frac{b_p - b_f}{5} \tag{10.36}$$

where F_{up} = the specified tensile strength of the bearing plate and V_{cs} = the horizontal shear force carried by the concrete strut.

Where split FBPs are used, the plate height, d_p, should not be less than $0.45d_w$.

Compression field. The nominal strength of the concrete compression field mechanism, V_{cfn}, is calculated as follows:

$$V_{cfn} = V_c' + V_s' \leq 1.7\sqrt{f_c'}b_o h \tag{10.37}$$

$$V_c' = 0.4\sqrt{f_c'}b_o h \tag{10.38}$$

$$V_s' = \frac{A_{sh}F_{ysh}\,0.9h}{s_h} \tag{10.39}$$

where f_c' = the concrete compressive strength, in MPa
V_c' = the concrete contribution to nominal compression field, $V_c' = 0$ where the column is in tension
V_s' = the contribution provided by the horizontal ties to nominal compression field strength
A_{sh} = the cross-sectional area of reinforcing bars in each layer of ties spaced at s_h through the beam depth, $A_{sh} \geq 0.004bs_h$

Where extended FBP and/or steel columns are used, they should be designed to resist a force equal to the joint shear carried by the outer compression field, V_{cf}. The thickness of column flanges or the extended FBP is considered adequate if the following equation is satisfied:

$$t_f \geq 0.12\sqrt{\frac{V_{cf}b_p'}{d_o F_y}} \tag{10.40}$$

where V_{cf} = the horizontal shear force carried by the outer compression field
b_p' = the flange width of the steel column or the width of the extended FBP
F_y = the specified yield strength of the plate

In addition to the preceding requirement, the thickness of the extended FBP should not be less than the thickness of the FBP between the beam flanges.

Ties above and below the beam should be able to transfer the force, V_{cf}, from the beam flanges into the outer concrete panel. In addition to the requirements in Sec. 10.4.6.1, the minimum total cross-sectional area should satisfy the following:

$$A_{\text{tie}} \geq \frac{V_{cf}}{F_{ysh}} \tag{10.41}$$

where F_{ysh} = the yield strength of the reinforcement
A_{tie} = the total cross-sectional area of ties located within the vertical distance $0.4d$ of the beam (see Fig. 10.31)

10.4.6.3 Vertical column bars. To limit the slip of column bars within the joint, the size of the bar should satisfy the following requirements:

$$d_b < \frac{d + 2d_o}{20} \tag{10.42}$$

where, for single bars, d_b = the vertical bar diameter, and, for bundled bars, d_b = the diameter of a bar of equivalent area to the bundle.

Exceptions to Eq. (10.42) can be made where it can be shown that the change in force in vertical bars through the joint region, ΔF_{bar}, satisfies the following:

$$\Delta F_{\text{bar}} < 80(\mathrm{d} + 2d_{\mathrm{o}})\sqrt{f_c'} \tag{10.43}$$

where f_c' is in MPa.

10.4.7 Limitations

The ASCE recommendations are limited to joints where the steel beams are continuous through the reinforced concrete column. Although this type of detail has been successfully used in practice, the guidelines do not intend to imply or recommend the use of this type over other possible details. Both interior and exterior joints can be designed using the recommendations; however, top-interior and top-corner joints are excluded because supporting test data are not available. For earthquake loading, the recommendations are limited to regions of low-to-moderate seismic zones. The ratio of depth of concrete column, *h,* to the depth of the steel beam, *d,* should be in the range of 0.75 to 2.0. For the purpose of strength calculation, the nominal concrete strength, f_c', is limited to 40 MPa (6 ksi) and only normal-weight concrete is allowed, the reinforcing bars yield stress is limited to 410 MPa (60 ksi), and the structural steel yield stress is limited to 345 MPa (50 ksi).

10.5 Connections to Concrete-Filled Tube (CFT) Columns

10.5.1 Introduction

Steel tubes of relatively thin wall thickness filled with high-strength concrete have been used in building construction in the United States and far east Asian countries. This structural system allows the designer to maintain manageable column sizes while obtaining increased stiffness and ductility for wind and seismic loads. Column shapes can take the form of tubes or pipes as required by architectural restrictions. Additionally, shop fabrication of steel shapes helps ensure quality control.

In this type of construction, in general, at each floor level a steel beam is framed to these composite columns. Often, these connections are required to develop shear yield and plastic moment capacity of the beam simultaneously.

10.5.2 Current practice

In current practices, there are very limited guidelines for selecting or designing connections for attaching steel beams to CFT columns. In these instances, heavy reliance is made on the judgment and experience of individual designers.

The majority of available information on steel beams to CFT columns has been developed as a result of the U.S.–Japan Cooperative Research Program on Composite/Hybrid Structures (1992). It should be noted that the information developed under this initiative is targeted toward highly seismic regions. Nevertheless, the information could be used to design connection details in nonseismic regions.

One of the distinct categories of connection details suggested is attaching the steel beams using full-penetration welds, as practiced in Japan. Japanese practice usually calls for a massive amount of field and shop welding. Figure 10.33 shows some of the connection details suggested in Japan. In general, the type of details that are used in Japan are not economical for U.S. practice.

10.5.3 Problems associated with welding beams to CFT columns

When beams are welded or attached to steel tubes through connection elements, complicated stiffener assemblies are required in the joint area within the column. However, welding of the steel beam or connecting element directly to the steel tube of composite columns could produce potential problems, some of which are outlined in the following:

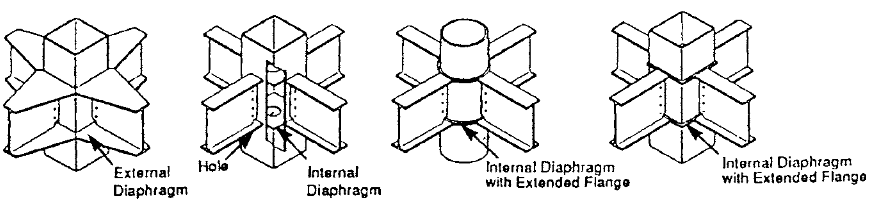

Figure 10.33 Typical connection details suggested in Japan.

1. Transfer of tensile forces to the steel tube can result in separation of the tube from the concrete core, thereby overstressing the steel tube. In addition, the deformation of the steel tube will increase connection rotation, decreasing its stiffness. This is especially important if the connection is required to develop full plastic moment capacity of the beam.
2. Welding of the thin steel tube results in large residual stresses because of the restraint provided by other connection elements.
3. The steel tube is designed primarily to provide lateral confinement for the concrete. Further, in building construction where CFT columns are utilized, the steel tube portion of the column also acts as longitudinal reinforcement. Transferring additional forces from the beam to the steel tube could result in overstressing the steel tube portion of the column.

10.5.4 Possible connection detail

With these considerations in mind, Azizinamini and Parakash (1993) and Azizinamini et al. (1995) suggest two general types of connections. Figure 10.34 shows one alternative in which forces are transmitted to

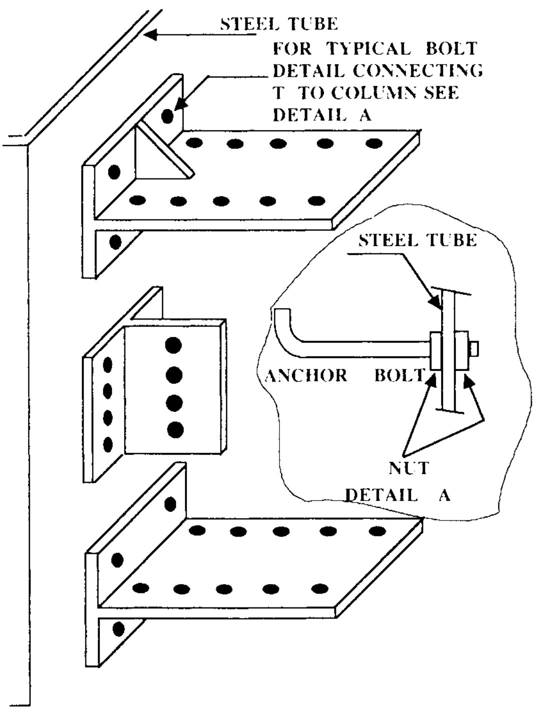

Figure 10.34 Connection detail using anchor bolts.

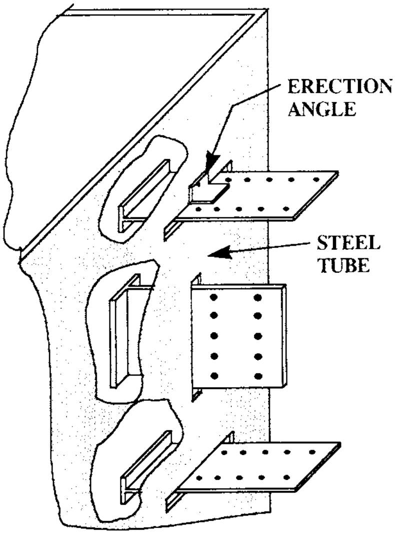

Figure 10.35 Connection detail using embedded elements.

the core concrete via anchor bolts connecting the steel elements to the steel tube. In this alternative, all elements could be preconnected to the steel tube in the shop. The nut inside the steel tube is designed to accomplish this task. The capacity of this type of connection would be limited to the pull-out capacity of the anchor bolts and local capacity of the tube.

Another variation of the same idea is shown in Fig. 10.35, where connecting elements would be embedded in the core concrete via slots cut in the steel tube. In this variation, slots must be welded to connection elements after beam assembly for concrete confinement. The ultimate capacity of this detail also would be limited to the pull-out capacity of the connection elements and the concrete in the tube. The types of connections shown in Figs. 10.34 and 10.35 could be suitable to nonseismic applications, at the story levels, where the level of forces is relatively small.

Another suggested type of connection (Azizinamini and Parakash, 1993; Azizinamini et al., 1995) is to pass the beam completely through the column, as shown in Fig. 10.36. In this type of detail, a certain height of column tube, together with a short beam stub passing through the column and welded to the tube, could be shop-fabricated to form a "tree column" as shown in Fig. 10.37. The beam portion of the tree column could then be bolted to girders in the field.

Alostaz and Schneider (1996) report tests on six different connection details for connecting steel beams to circular CFT columns. The objectives of these tests were to examine the feasibility of different connection details for use in highly seismic areas and suitable to U.S. practice.

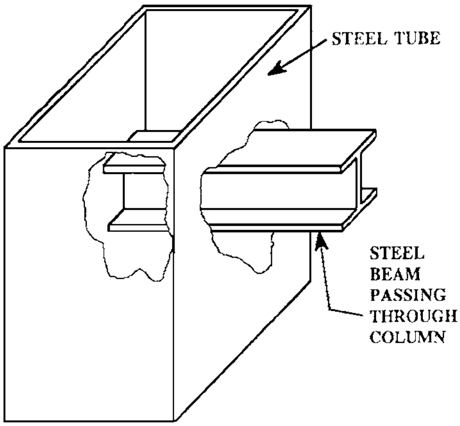

Figure 10.36 Through-connection detail.

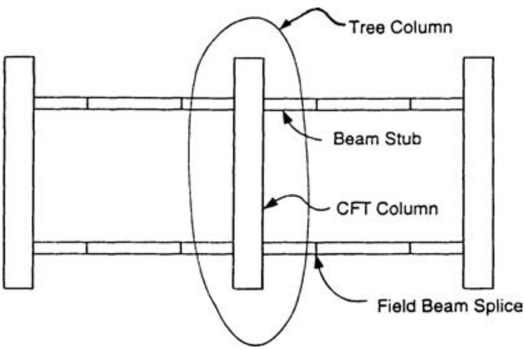

Figure 10.37 Tree column construction concept.

These connections ranged from a very simple detail that attached the beam to the tube skin as in connection type I to a more rigid detail in which the girder was passed through the tube core as represented by connection type VII. All connections were designed with a beam stub. The beams were bolted and/or welded to these stubs. The specimens had a T configuration, thus representing an exterior joint in a building. Each specimen consisted of a 14- × ¼-in (356- × 6.4-mm)-diam pipe and W 14 × 38 beam. The concrete compressive strength varied between 7.8 and 8.3 ksi (53.8 and 57.2 MPa). The pipe yield strength was 60 ksi (420 MPa). The stub flanges and web yield strengths were 50 and 40 ksi (350 and 280 MPa), respectively. This resulted in a column-to-beam bending capacity ratio of approximately 2.6. This relatively high column-to-beam capacity ratio is not desirable when one attempts to investigate connection behavior. At the extreme, very high column moment capacity will force the plastic hinge to form at the end of the beam, preventing the investigation of behavior of joints. Despite this shortcoming, Alostaz and Schneider's data provide valuable information that could be used to develop connection details suitable for seismic as well as nonseismic applications. Following is a brief discussion of the behavior of different connection details tested by Alostaz and Schneider (1996).

10.5.4.1 Simple connection, type I. Figure 10.38 illustrates the details of this specimen. The flange and web plates of the connection stub were welded directly to the steel pipe. At the tube face, the flange plates were flared to form a central angle of 120°, and the width of the plates was decreased gradually over a 10-in (254-mm) distance to match that of the girder flanges. Figure 10.39 shows the load-displacement relationship. Failure was due to fracture at the flange tip on the connection stub and pipe wall tearing. The connection survived a limited number of inelastic cycles and it could not develop the plastic flexural strength of the girder. This connection had the lowest flexural strength and was the most flexible of all connections tested. This connection had a ductility ratio of 1.88, which was the lowest of all connections tested. The flexural ductility ratio (FDR) was defined as

$$\mathrm{FDR} = \frac{\delta_{\mathrm{max}}}{\delta_{\mathrm{yield}}} \qquad (10.44)$$

where δ_{max} is the maximum displacement at the girder tip prior to failure and δ_{yield} is the yield displacement obtained experimentally.

10.5.4.2 Continuous web plate connection, type IA. In an attempt to improve the behavior of connection type I, the web plate was extended through the concrete core. To continue the web through the tube, a vertical slot was cut on opposite sides of the tube wall. The web plate was fillet-welded to the tube. Figure 10.40 illustrates the details of this specimen. Figure 10.41 shows the load-displacement relationship. The hysteretic behavior of this modified connection exhibited significant improvement compared to the original simple connection. This connection was able to develop approximately 1.26 times the flexural plastic strength of the girder and the initial stiffness was comparable to the ideal rigid connection. However, the strength deteriorated rapidly and only 50% of the girder bending strength remained at the end of the test. This connection had a ductility ratio of 2.55.

10.5.4.3 Connection with external diaphragms, type II. Behavior of the simple connection was improved by expanding the connection stub flanges to form external diaphragms. The diaphragm was fillet-welded to the pipe wall on both sides of the plate. Figure 10.42 illustrates the details of this specimen. Figure 10.43 shows the load-displacement relationship. The hysteretic performance of this connection improved

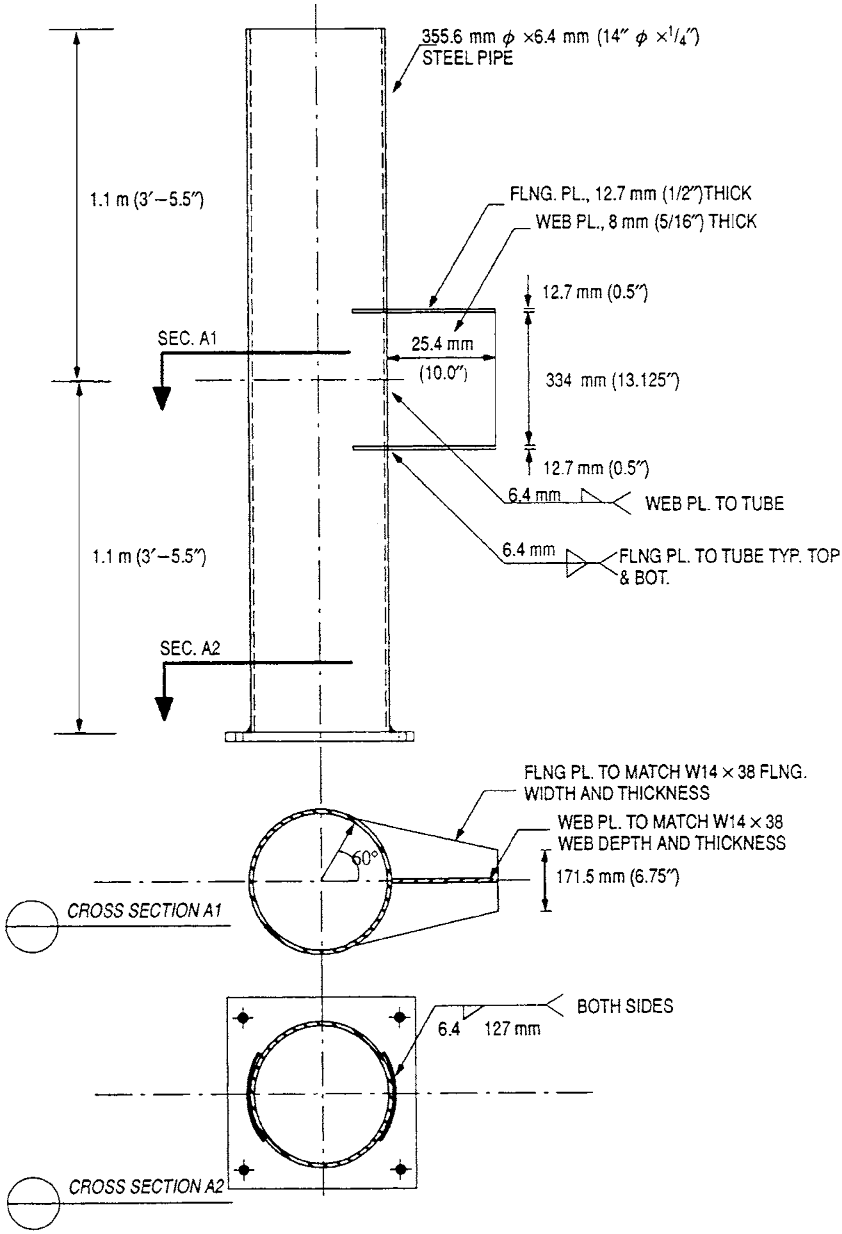

Figure 10.38 Simple connection, type I (*Alostaz and Schneider, 1996*).

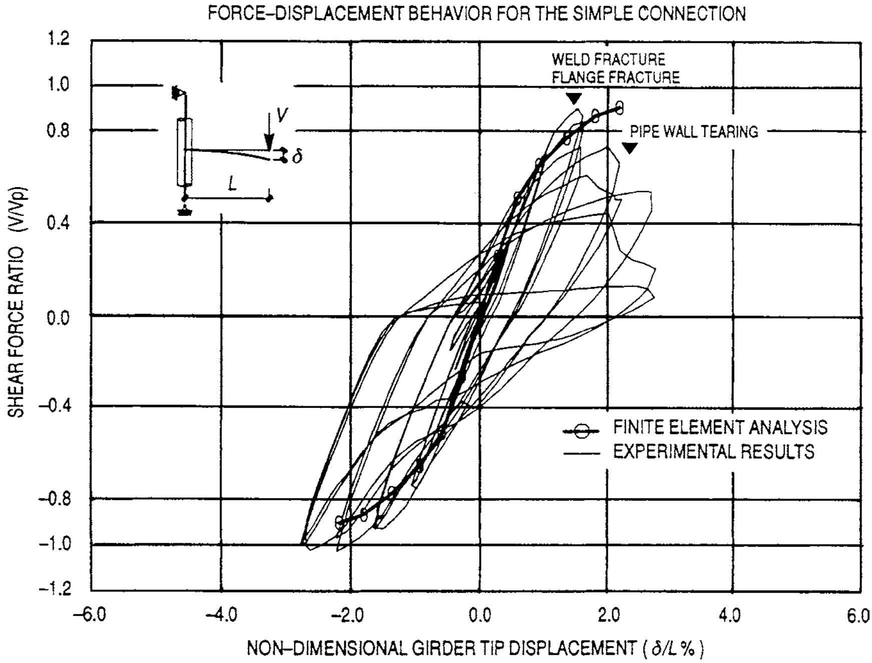

Figure 10.39 Load-displacement behavior of connection type I (*Alostaz and Schneider, 1996*).

relative to the simple connection type I. This resulted in a connection strength of approximately 17% higher than the girder bending strength. The geometry of the diaphragm was a critical issue in the behavior of this detail. The sharp reentrant corner between the diaphragm and the girder created a large stress concentration which initiated fracture in the diaphragm. This fracture caused rapid deterioration in the connection performance. Significant tearing was noted through the welded region of the diaphragm plates. Although connection type IA had higher strength, its strength deteriorated at a faster rate compared to the connection with external diaphragms. This connection had a ductility ratio of 2.88. Analytically, this detail exhibited significant improvement when the girder was shifted further away from the CFT column face.

10.5.4.4 Connection with deformed bars, type III. This specimen is identical to connection type I, except that holes were drilled in the pipe to insert weldable deformed bars into the core of the tube. Four #6 (19-mm) deformed bars were welded to each flange. Figure 10.44 illustrates the details of this specimen. Figure 10.45 shows the force-displacement relationship. This connection exhibited stable strain-

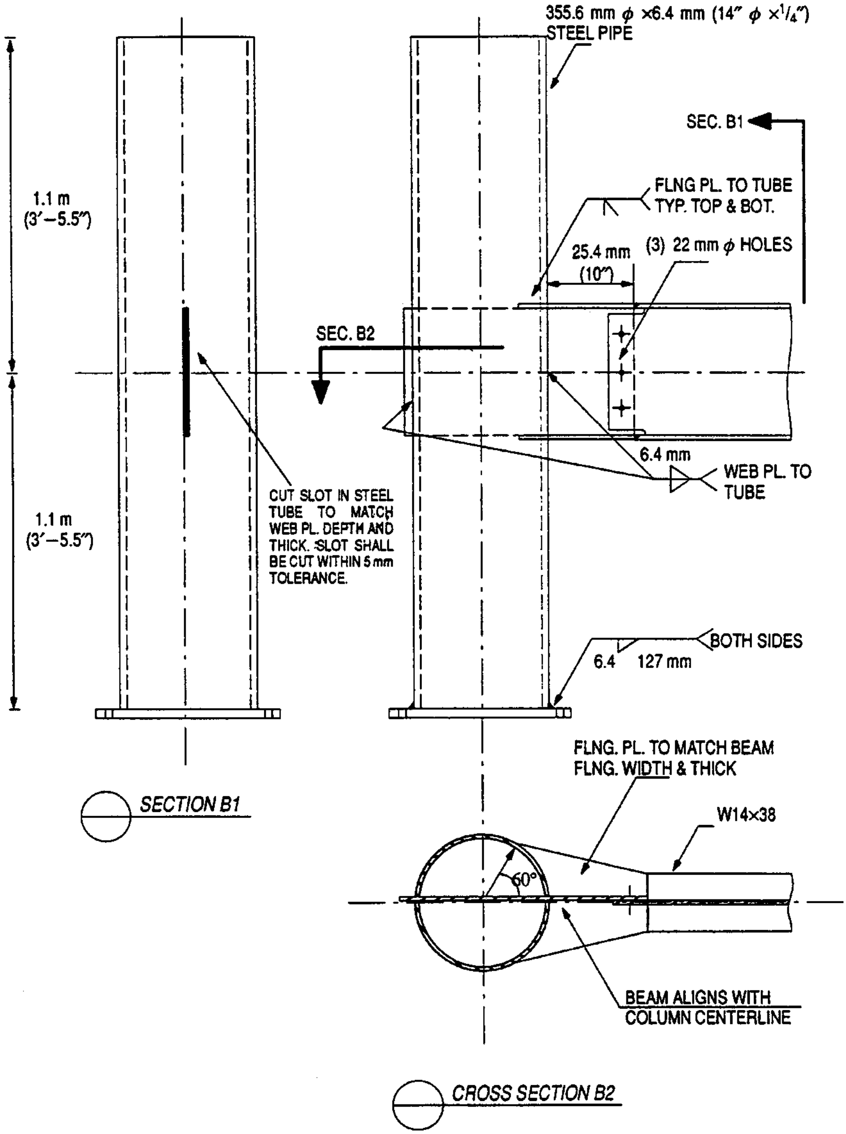

Figure 10.40 Simple connection with continuous web plate, type IA (*Alostaz and Schneider, 1996*).

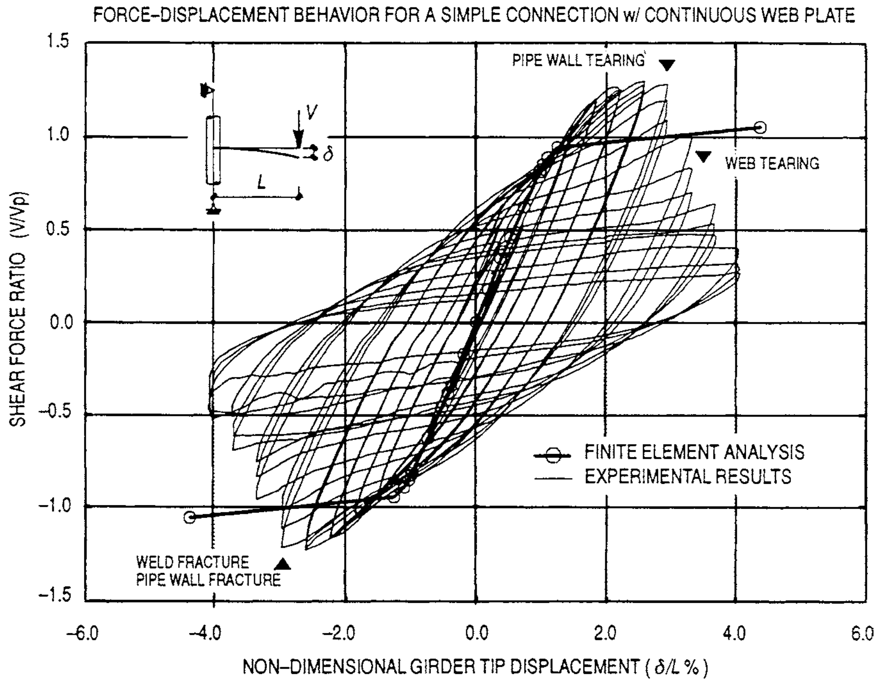

Figure 10.41 Load-displacement behavior of connection type IA (*Alostaz and Schneider, 1996*).

hardening behavior up to failure, and it developed approximately 1.5 times the girder bending strength. Failure was sudden and occurred by rupture of three of the four deformed bars in the connection detail, while the fourth bar failed by pull-out of the concrete core. The connection ductility was approximately 3.46 compared to only 1.88 for an identical connection without the deformed bars. The clearance, weldability of the deformed bars, and the configuration of the weld on the bars are critical issues in this detail.

10.5.4.5 Continuous flanges, type VI. To resolve the problems of connection type III, the connection stub flanges were continued through the pipe and fillet-welded to the pipe wall. A shear tab was fillet-welded to the tube skin. No effort was made to enhance the bond between the embedded flanges and the concrete core. Figure 10.46 illustrates the details of this specimen. Figure 10.47 shows the force-displacement relationship. The fillet weld attaching the flanges to the tube wall fractured at low amplitude cyclic deformations. The embedded flanges slipped through the concrete core without significant resistance. The hysteretic curves were quite pinched and it is likely that this connection may not perform well during a severe seismic event.

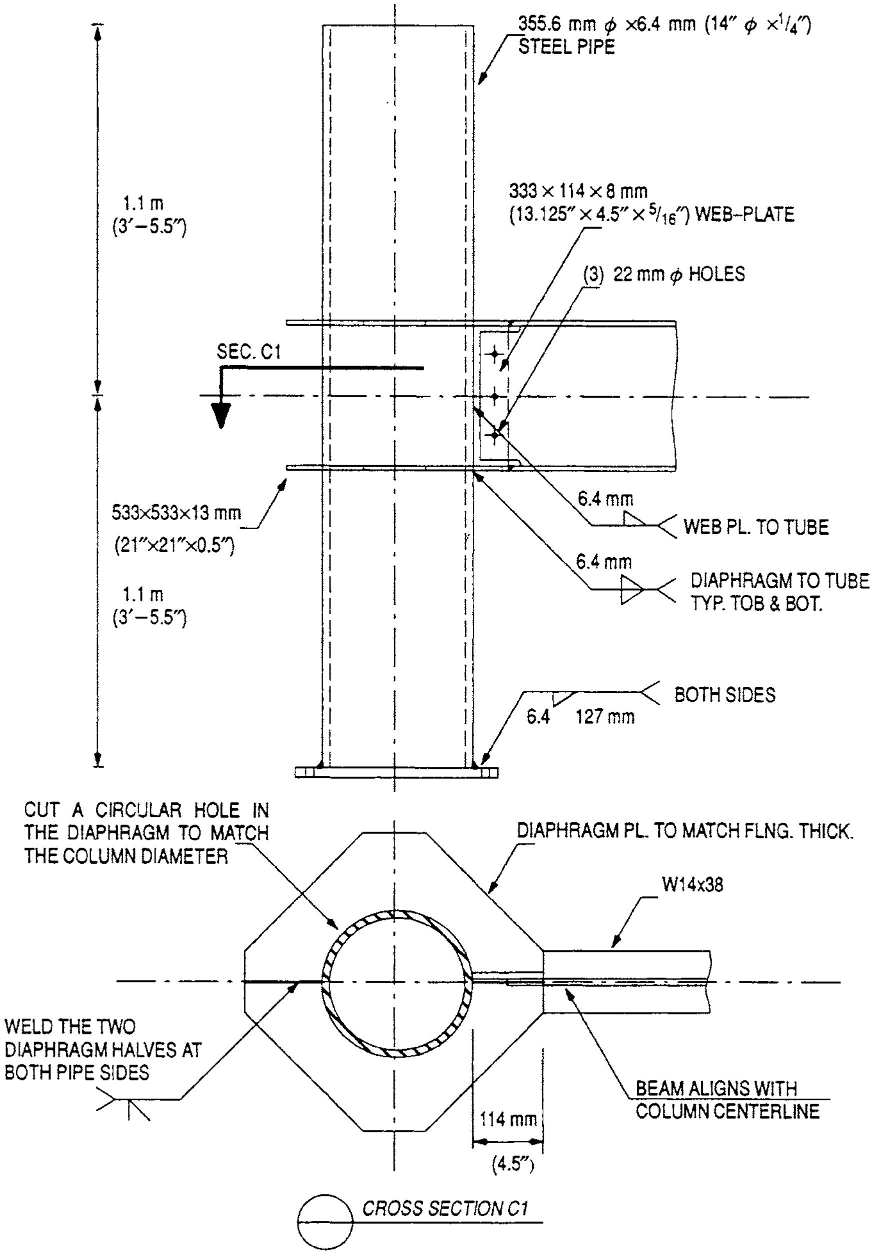

Figure 10.42 Connection with external diaphragms, type II (*Alostaz and Schneider, 1996*).

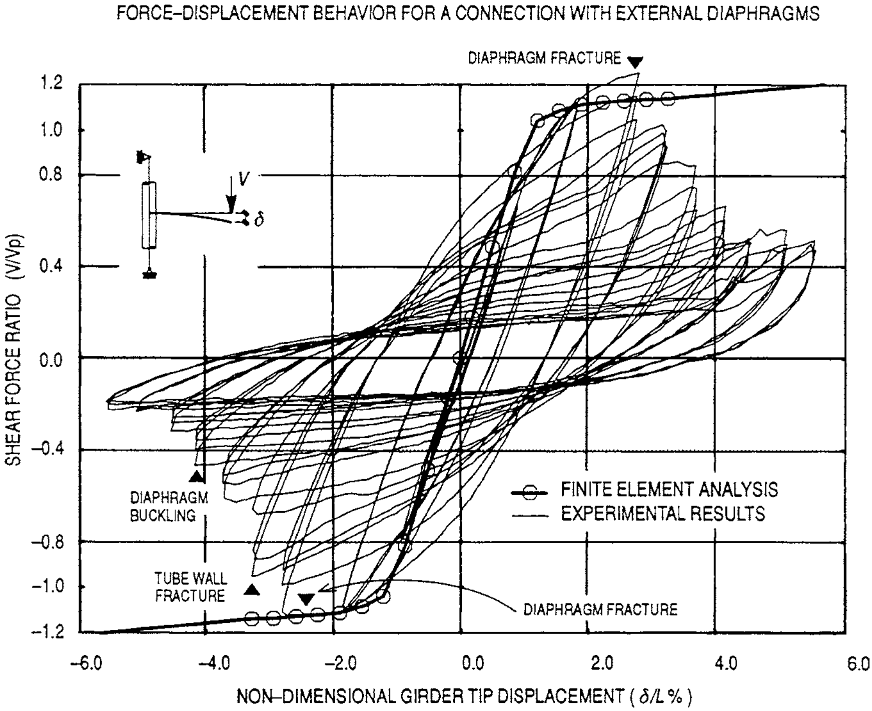

Figure 10.43 Load-displacement behavior of connection type II (*Alostaz and Schneider, 1996*).

10.5.4.6 Through-beam connection detail, type VII. Alostaz and Schneider (1996) also tested one specimen with the through-beam connection detail suggested by Azizinamini and Parakash (1993) and Azizinamini et al. (1995). In this detail, the full cross section of the girder was continued through the tube core. An I-shaped slot was cut in the tube wall and the beam stub was passed through the pipe. The beam stub was fillet-welded to the pipe. Figure 10.48 illustrates the details of this specimen. Figure 10.49 shows the force-displacement relationship. The flexural strength of this connection exceeded 1.3 times the plastic bending strength of the girder. This detail had a ductility ratio of 4.37, the highest of all connections tested. It also had a satisfactory hysteretic performance. Table 10.1 shows a summary of the flexural characteristics of the tested connections.

Results of Alostaz and Schneider tests indicated that the through-beam connection detail had the best performance, especially for seismic regions.

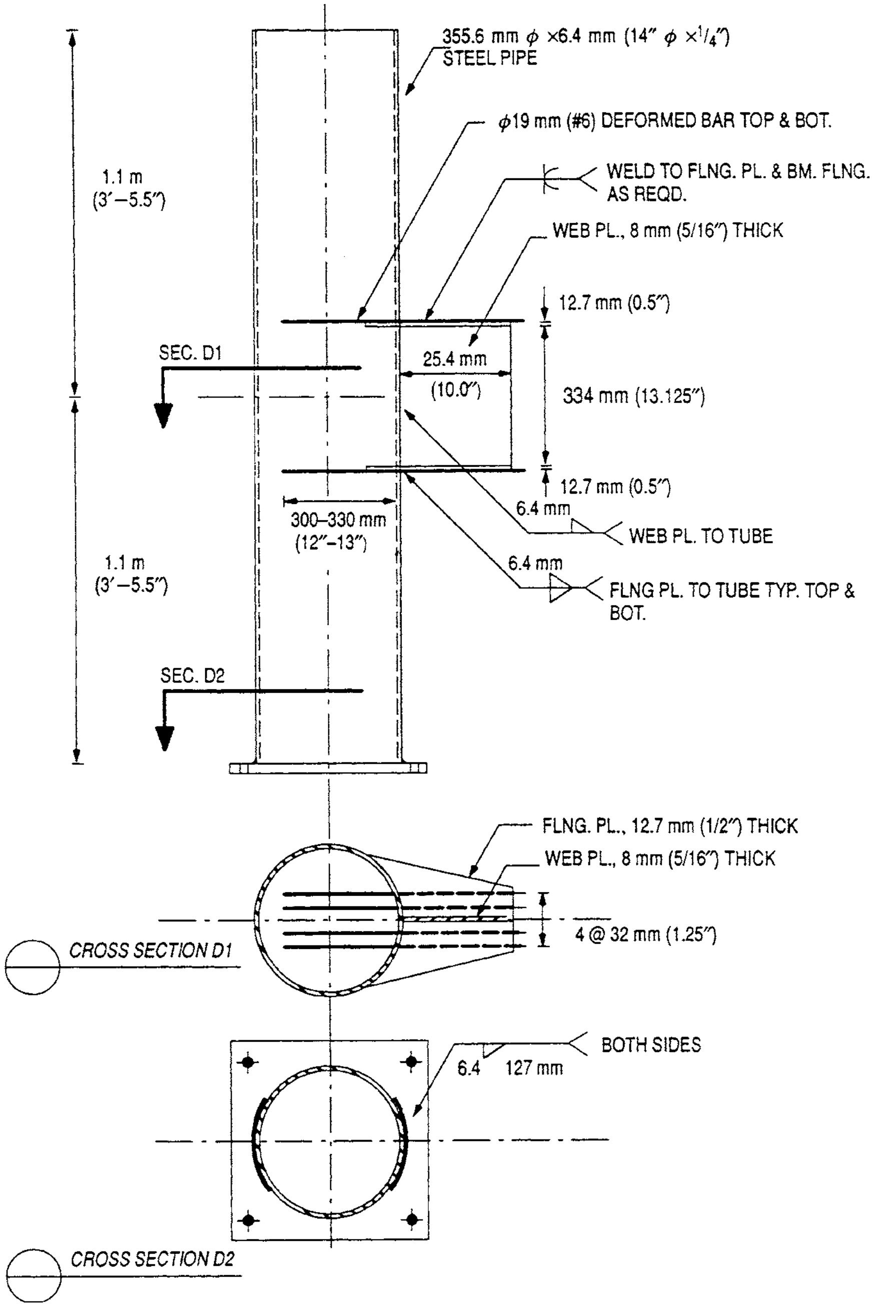

Figure 10.44 Connection with embedded deformed bars, type III (*Alostaz and Schneider, 1996*).

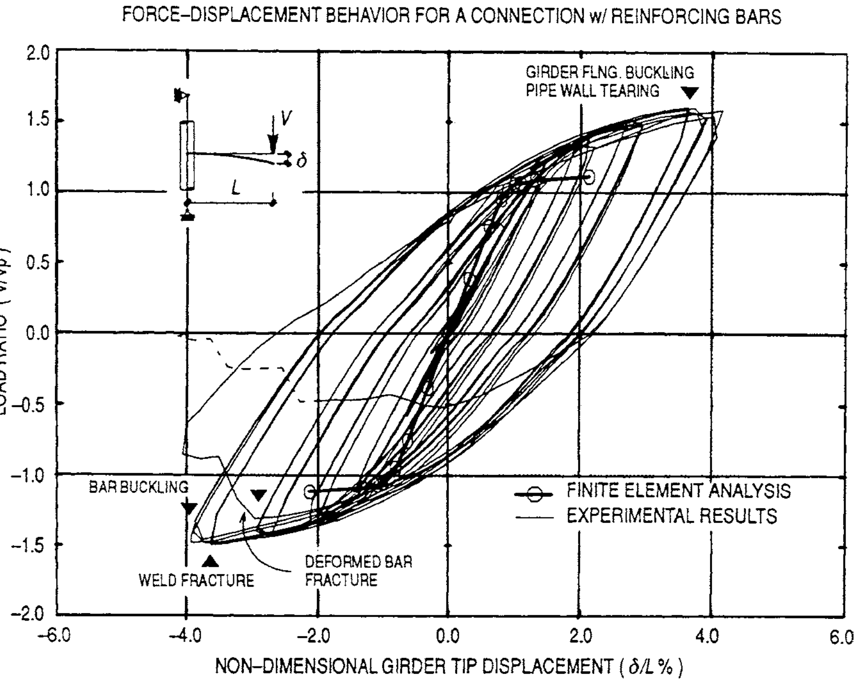

Figure 10.45 Load-displacement behavior of connection type III (*Alostaz and Schneider, 1996*).

10.5.4.7 Other connection details. Ricles et al. (1997) report results of cyclic tests conducted on beams attached to rectangular CFT columns using bolted or welded tees. Figures 10.50 through 10.52 show connection details for three of the specimens tested (specimens C4, C5, and C6). The split tees in these specimens were posttensioned to the column using 14-A490 bolts after curing of the concrete. These bolts were passed through the column using PVC conduits placed prior to casting concrete. In specimens C4 and C5, 22-mm-diam A325 bolts with 2-mm oversized bolt holes were used to attach the beam flanges to split tees. In specimen C6, however, 12-mm fillet welds were used to attach the beam flanges to split tees. In specimens C5 and C6, the shear tabs for attaching the beam web to CFT column were omitted.

Figures 10.53 through 10.55 give plots of applied beam moment versus the resulting plastic rotation at the connection level for the three test specimens. These specimens were designed based on AISC LRFD seismic provisions following the weak beam-strong column configuration.

Test observations indicated that damage to the joint area was eliminated. Some elongation of A490 bolts was observed. This was attributed

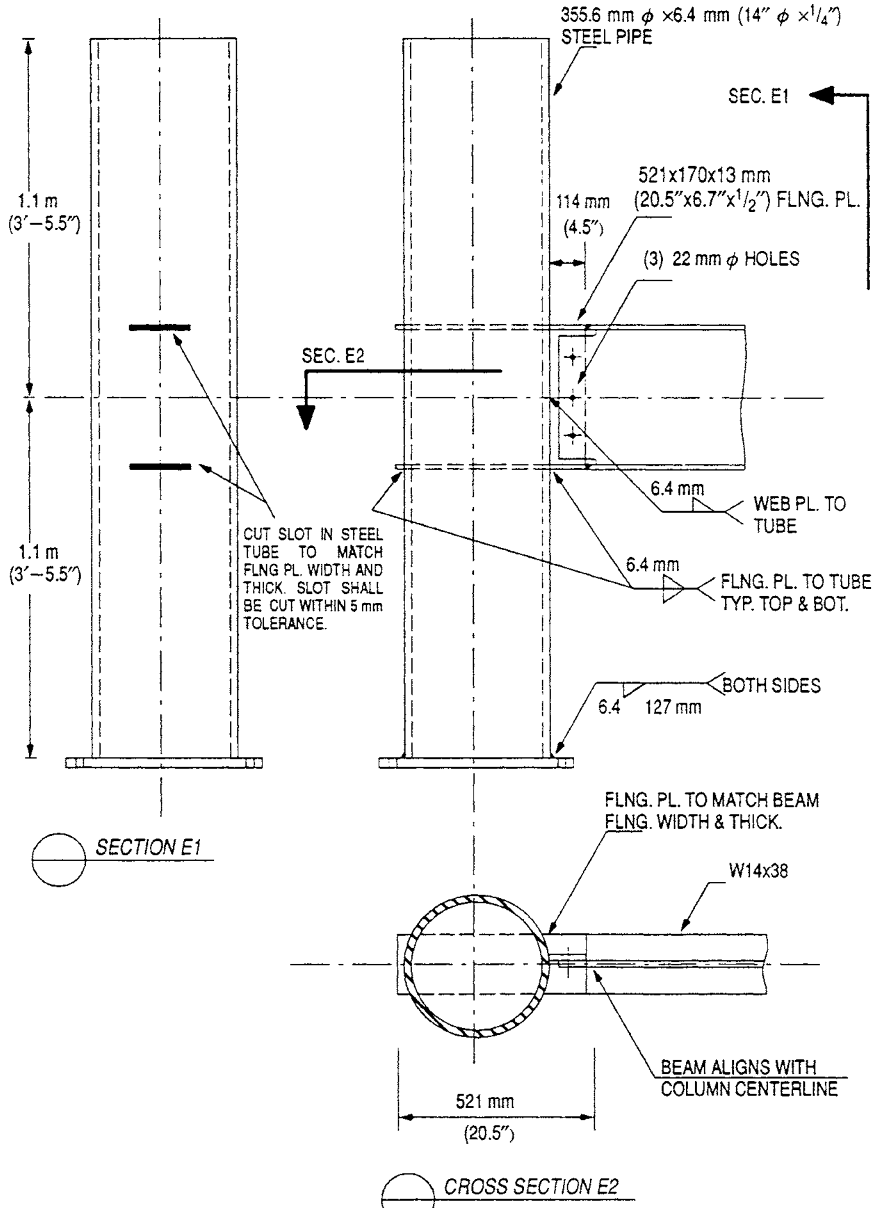

Figure 10.46 Continuous flanges, type VI (*Alostaz and Schneider, 1996*).

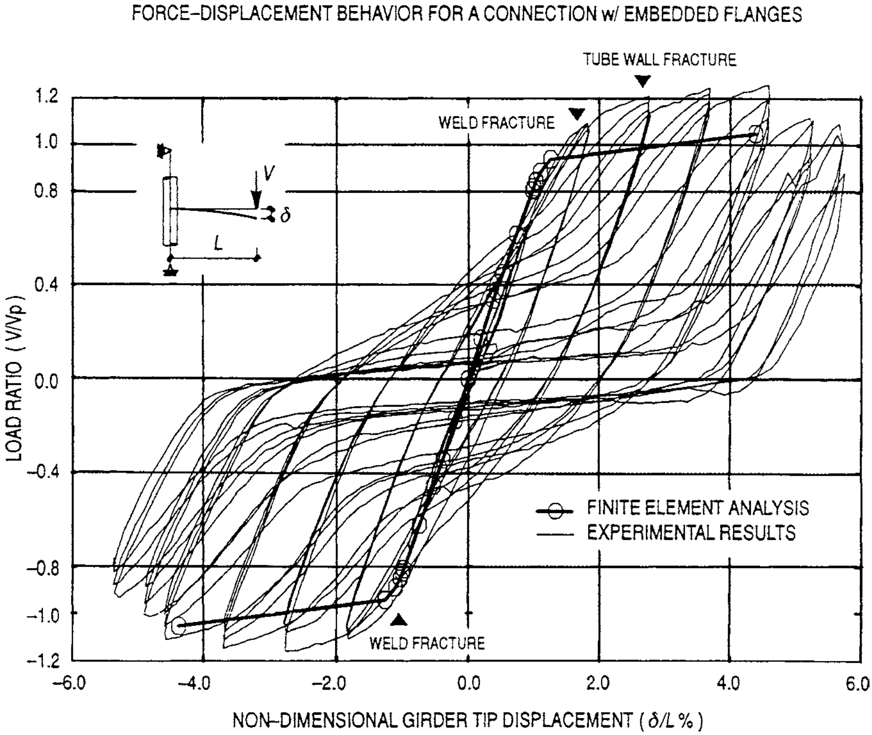

Figure 10.47 Load-displacement behavior of connection type VI (*Alostaz and Schneider, 1996*).

to compressive bearing forces transferred from split tees to CFT columns, causing distortion of the joint area in CFT columns. Another major observation was the slippage of the stem of split tees with respect to beam flanges in specimens C4 and C5. Ricles et al. (1997) were able to eliminate this slippage by welding washers to the beam flanges. The washers, acting as reinforcing material around the bolt hole, prevented bolt hole elongation and elimination of the slippage.

In this type of detail, attention should be directed to shear transfer between the beam end and CFT column. The load path for transferring the beam shear force to the CFT column is as follows. The beam end shear is first transferred as axial force from the beam end to the steel tube portion of the CFT column. This axial compressive or tensile force could only be transferred to the concrete portion of the CFT column if composite action between the steel tube and the concrete core exists. There are several ways through which this composite action could be developed. Friction due to bending or use of shear studs are two possible mechanisms. The guidelines for such shear-transfer mechanisms

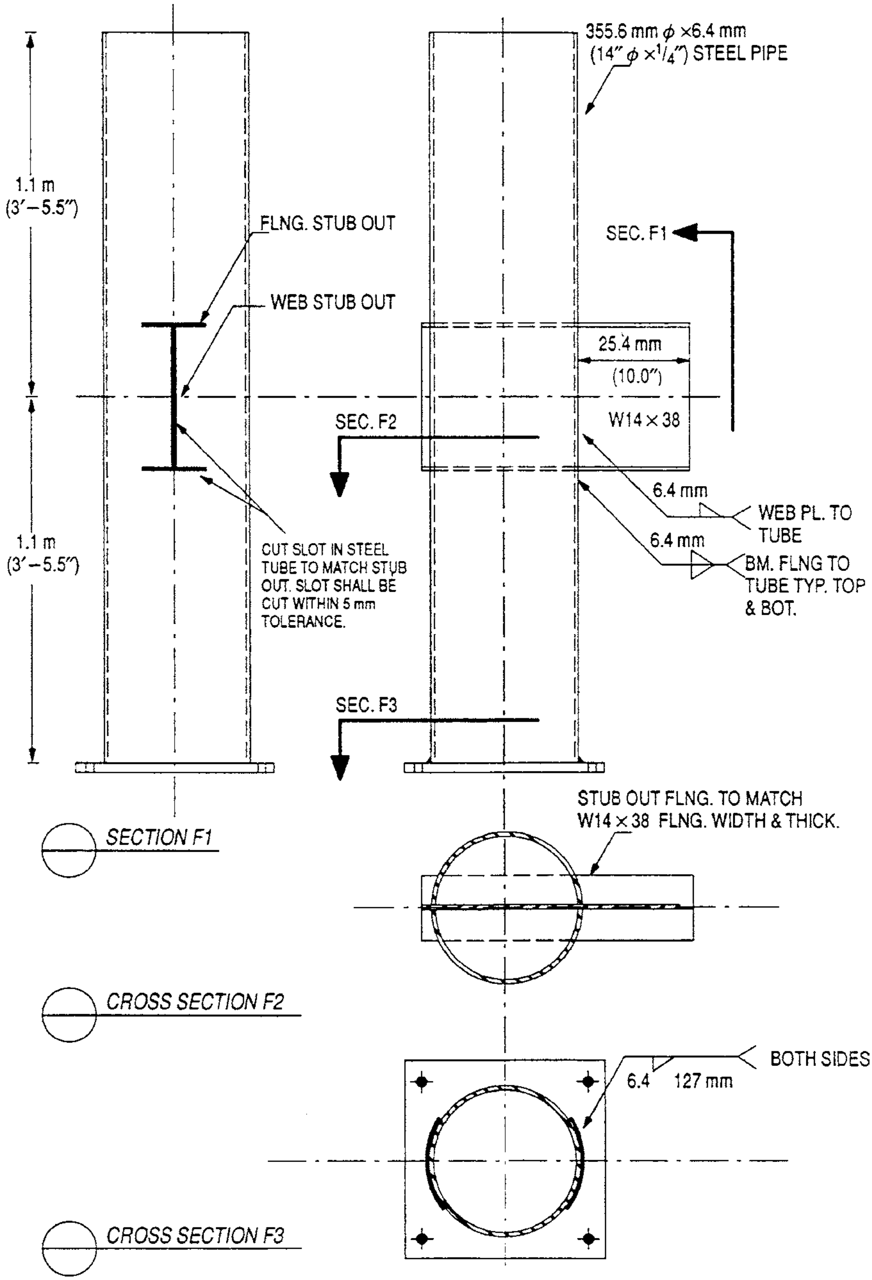

Figure 10.48 Continuation of the girder through the column, type VII (*Alostaz and Schneider, 1996*).

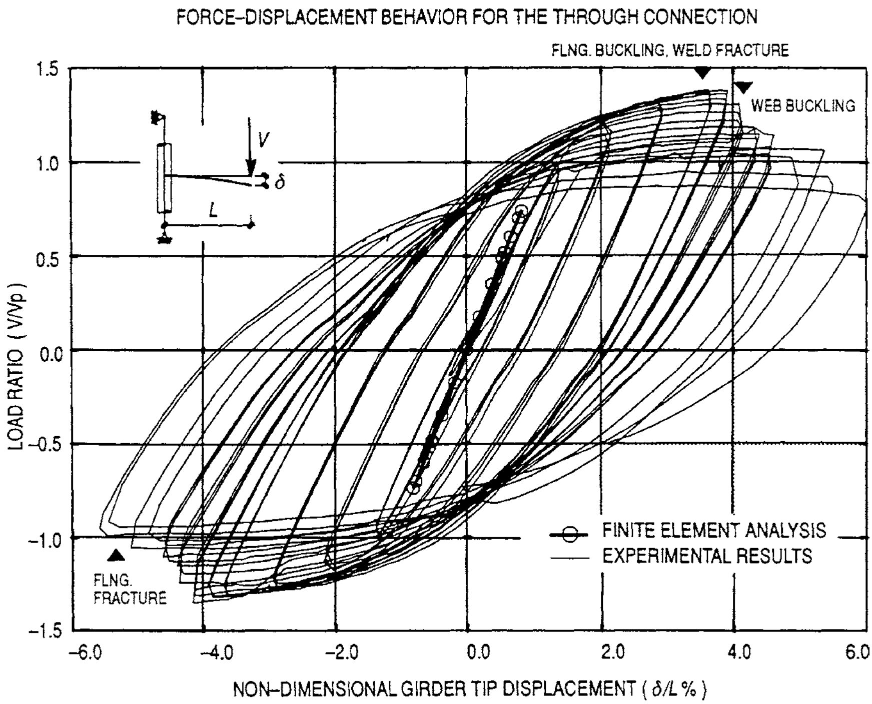

Figure 10.49 Load-displacement behavior of connection type VII (*Alostaz and Schneider, 1996*).

TABLE 10.1 Flexural Characteristics of the Tested Connections

Detail	Ductility	M_{max}/M_p	Initial stiffness ratio
I	1.88	0.97	85
IA	2.55	1.26	100
II	2.83	1.17	100
III	3.46	1.56	106
VI	3.76	1.23	100
VII	4.37	1.37	100

are still lacking. Ongoing research by Roeder (1997) attempts to resolve this issue.

10.5.5 Force transfer mechanism for through-beam connection detail

A combination of analytical and experimental investigations were undertaken to approximate the force transfer mechanism for the

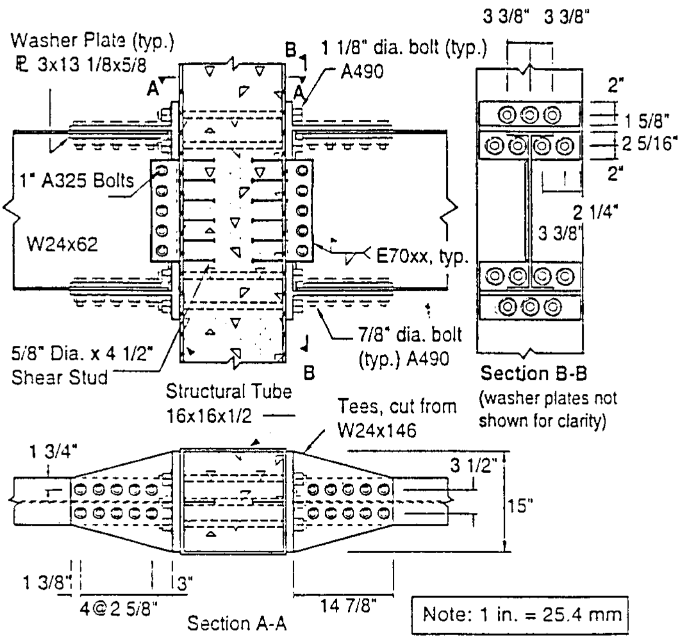

Figure 10.50 Split tee connection detail, specimen C4 (*Ricles et al., 1997*).

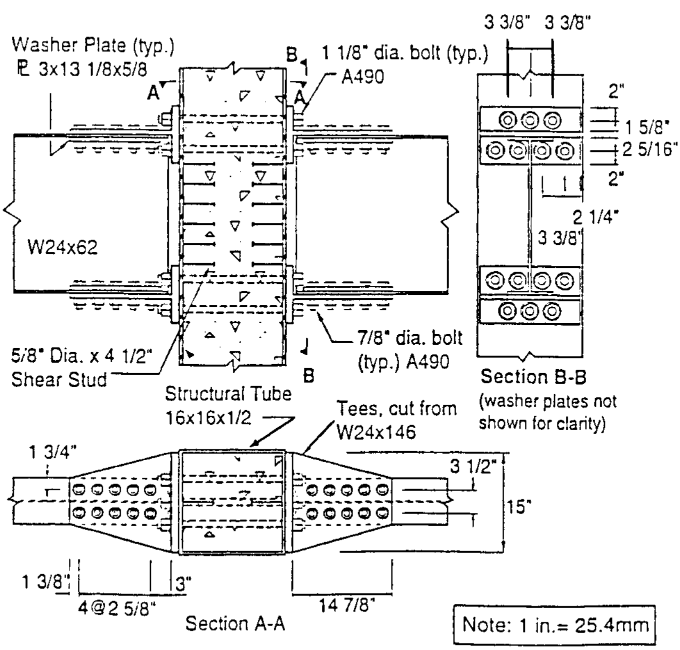

Figure 10.51 Split tee connection detail, specimen C5 (*Ricles et al., 1997*).

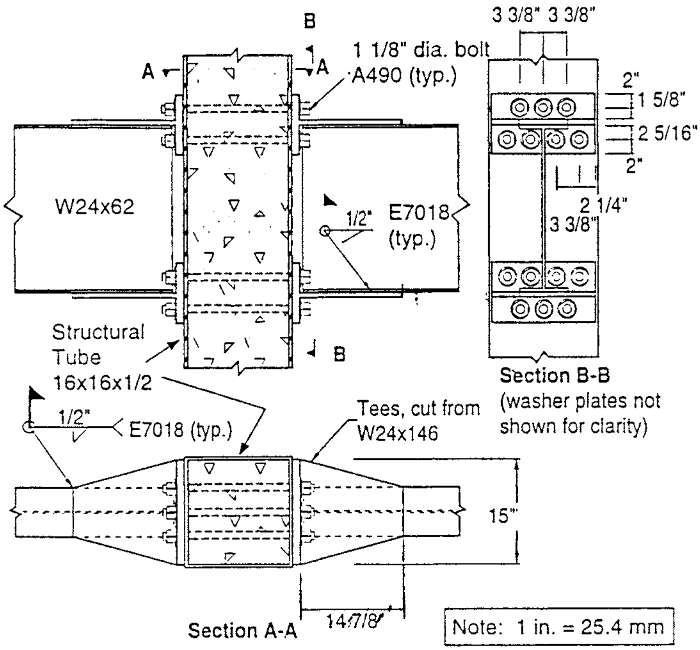

Figure 10.52 Split tee connection detail, specimen C6 (*Ricles et al., 1997*).

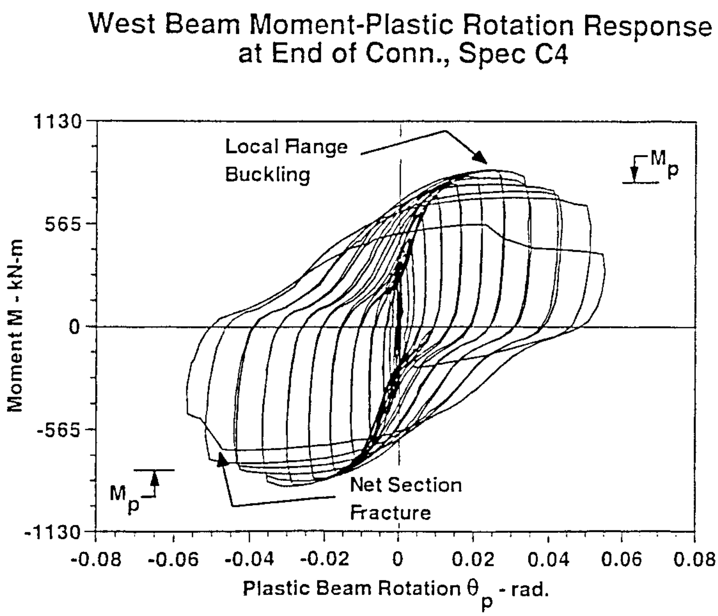

Figure 10.53 Moment-plastic rotation response, specimen C4 (*Ricles et al., 1997*).

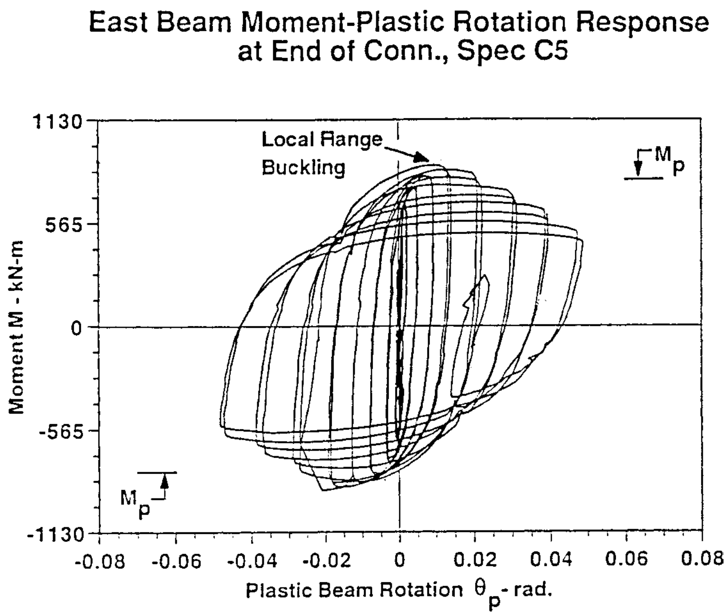

Figure 10.54 Moment-plastic rotation response, specimen C5 (*Ricles et al., 1997*).

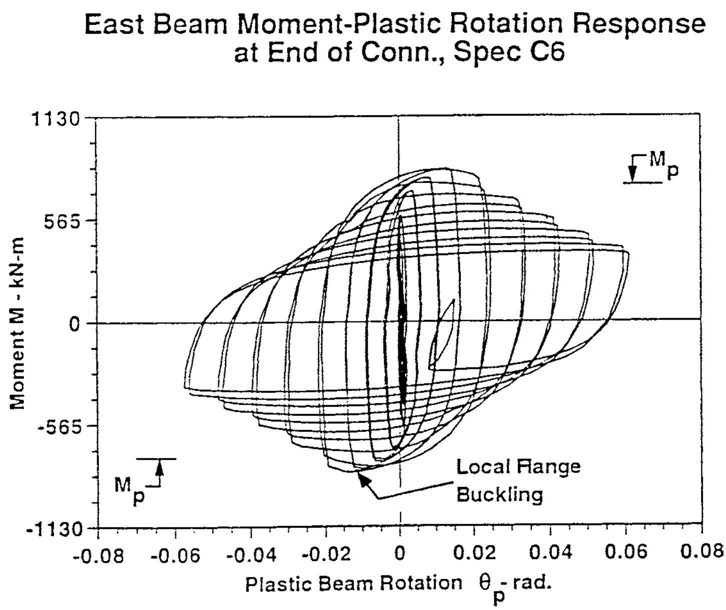

Figure 10.55 Moment-plastic rotation response, specimen C6 (*Ricles et al., 1997*).

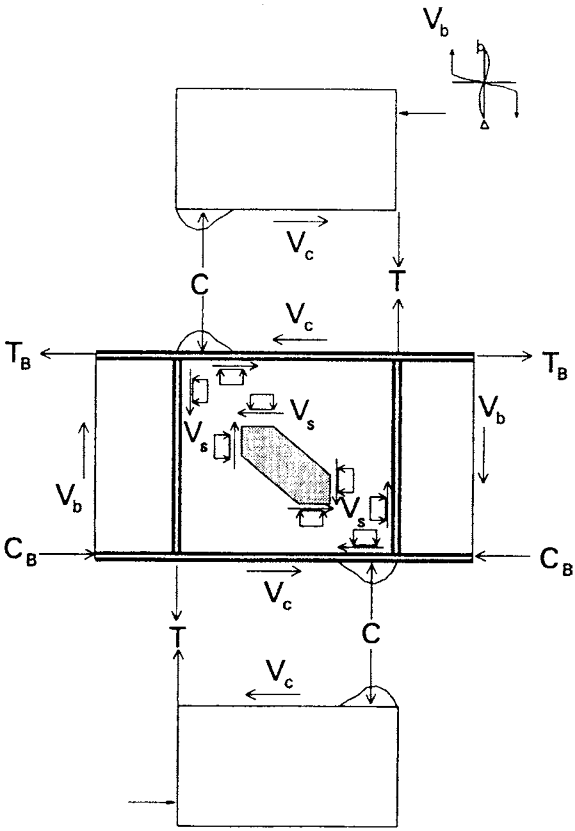

Figure 10.56 Force transfer mechanism for through-beam connection detail.

through-beam connection detail utilizing both circular and rectangular CFT columns (Azizinamini and Parakash, 1993; Azizinamini, et al., 1995).

Figure 10.56 shows the force transfer mechanism. A portion of the steel tube between the beam flanges acts as a stiffener, resulting in a concrete compression strut which assists the beam web within the joint in carrying shear. The effectiveness of the compression strut increases to a limit by increasing the thickness of the steel plate.

The width of the concrete compression strut on each side of the beam web in the direction normal to the beam web was approximately equal to half the beam flange width.

A compressive force block was created when beam flanges were compressed against the upper and lower columns (Fig. 10.56). The width of this compression block was approximately equal to the width of the beam flange. In the upper and lower columns, shown in Fig. 10.56, the compressive force C is shown to be balanced by the tensile force in the steel pipe. In Azizinamini and Parakash (1993), rods embedded in the concrete and welded to the beam flanges were provided to assist the steel tube in resisting the tensile forces and to minimize the tensile stresses in the steel tube. For small columns this may be necessary, however, for relatively larger columns there may not be a need for placing such rods. Ongoing research at the University of Nebraska—Lincoln is investigating this and other aspects of the force transfer mechanism. The next section suggests design provisions for the through-beam connection detail. These provisions are tentative and are applicable for both circular and rectangular CFT columns.

10.5.6 Tentative design provisions for through-beam connection detail

This tentative design procedure is in the form of equations relating the applied external forces to the connection details such as the thickness of the steel pipe. The design procedure follows the general guidelines in the AISC LRFD manual. In developing the design equations the following assumptions were made:

1. Externally applied shear forces and moments at the joints are known.
2. At the ultimate condition, the concrete stress distribution is linear and the maximum concrete compressive stress is below its limiting value.

The joint forces implied in assumption (1) could be obtained from analysis and require the knowledge of the applied shear and moment at the joint at failure. These quantities are assumed to be related as follows:

$$V_c = \alpha V_b$$
$$M_b = l_1 V_b$$
$$M_c = l_2 V_c$$

where V_b and M_b are the ultimate beam shear and moment, respectively, and V_c and M_c are the ultimate column shear and moment, respectively. Figure 10.57 shows these forces for an isolated portion of a structure subjected to lateral loads.

Assumption (2) is valid for the cases where the moment capacity of columns is relatively larger than the beam capacity.

Figure 10.58 shows the free-body diagram (FBD) of the beam web within the joint and upper column at ultimate load. With reference to Fig. 10.58 the following additional assumptions are made in deriving the design equations:

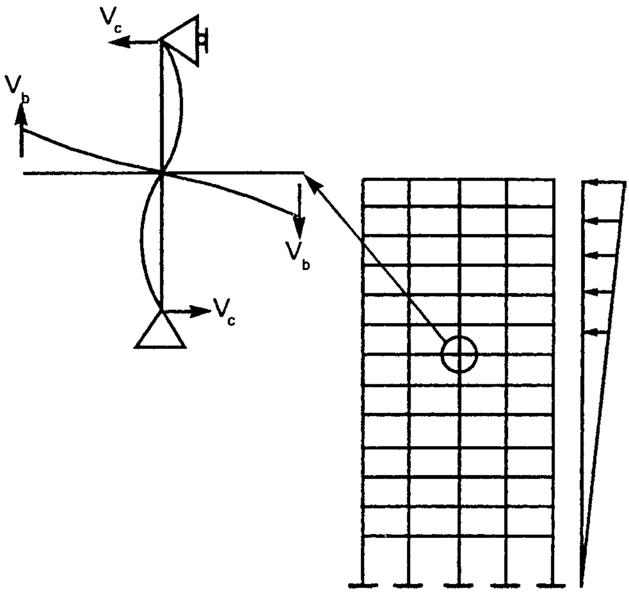

Figure 10.57 Assumed forces on an interior joint in a frame subjected to lateral loads.

1. The concrete stress distribution is assumed to be linear. The width of the concrete stress block is assumed to equal b_f, the beam flange width.
2. As shown in Fig. 10.58, the strain distribution over the upper column is assumed to be linear.
3. The steel tube and concrete act compositely.
4. The portion of the upper column shear, V_c, transferred to the steel beam is assumed to be ßC_c, where C_c is the resultant concrete compressive force bearing against the beam flange and ß is the coefficient of friction.
5. Applied beam moments are resolved into couples concentrated at beam flanges.
6. The resultant of the concrete compression strut is along a diagonal as shown in Fig. 10.58.

Considering the preceding assumptions and the strain distribution shown for the upper column in Fig. 10.58, the maximum strain in concrete, ε_c, could be related to ε_l, the steel pipe strain in tension:

$$\varepsilon_c = \frac{a}{d_c - a}\,\varepsilon_l \tag{10.45}$$

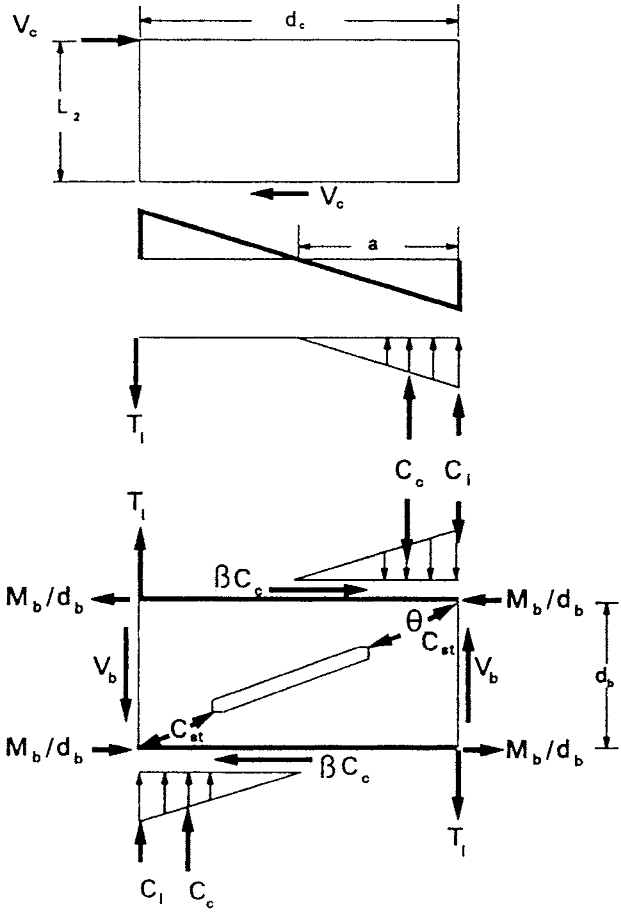

Figure 10.58 FBD of upper column and beam web within the joint.

The maximum stress in the concrete and the stresses in the steel tube can be calculated as follows:

$$f_c = E_c \varepsilon_c \tag{10.46}$$

$$f_{lc} = E_s \varepsilon_c \tag{10.47}$$

$$f_{lt} = E_s \varepsilon_l \tag{10.48}$$

where f_c, f_{lc}, and f_{lt} are the maximum concrete compressive stress, the stress in the steel pipe in compression, and the stress in the steel pipe in tension, respectively.

Substituting Eq. (10.45) into Eqs. (10.46) to (10.48) and multiplying Eqs. (10.46) to (10.48) by the corresponding area, the resultant forces for different connection elements can be calculated as follows:

$$C_c = \left(\frac{1}{2}\right)\left(\frac{1}{\eta}\right)\xi b_f \frac{a^2}{d_c - a} f_{yl} \tag{10.49}$$

$$C_l = \gamma\, \xi b_f t_l \frac{a}{d_c - a} f_{yl} \tag{10.50}$$

$$T_l = \xi\, \gamma b_f t_l f_{yl} \tag{10.51}$$

Using the FBD of the upper column, shown in Fig. 10.58, Eqs. (10.49) to (10.51), and satisfying the vertical force equilibrium, the following equation could be obtained:

$$t_l = \frac{a^2}{d_c - 2a} \frac{1}{2\gamma\eta} \tag{10.52}$$

where d_c = diameter of the steel tube
a = depth of neutral axis
η = ratio of modulus of elasticity for steel over modulus of elasticity of concrete
t_l = thickness of steel tube
γ = factor reflecting portion of steel tube effective in carrying tensile forces. Experimental data for square tubes indicated that it could be assumed that $\gamma = 2$. The same value is assumed for circular columns.

Considering the moment equilibrium of the FBD of the upper column shown in Fig. 10.58, the following expression can be derived:

$$\frac{1}{d_c - a}\left[\frac{a^3 d_c}{d_c - 2\mathrm{a}} + a^2\left(d_c - \frac{a}{3}\right)\right] = \frac{2\eta}{\xi} \frac{\alpha l_2}{b_f f_{yl}} V_b \tag{10.53}$$

where f_{yl} is the yield strength of the steel tube.

In Eq. (10.53), ξf_{yl}, is the stress level that the steel tube is allowed to approach at the ultimate condition. Based on the experimental data and until further research is conducted, it is suggested that a value of 0.75 be used for ξ.

Equations (10.52) and (10.53) relate the externally applied force, V_b, directly and the externally applied forces, V_c and M_c, indirectly (through the coefficients α and l_2) to different connection parameters.

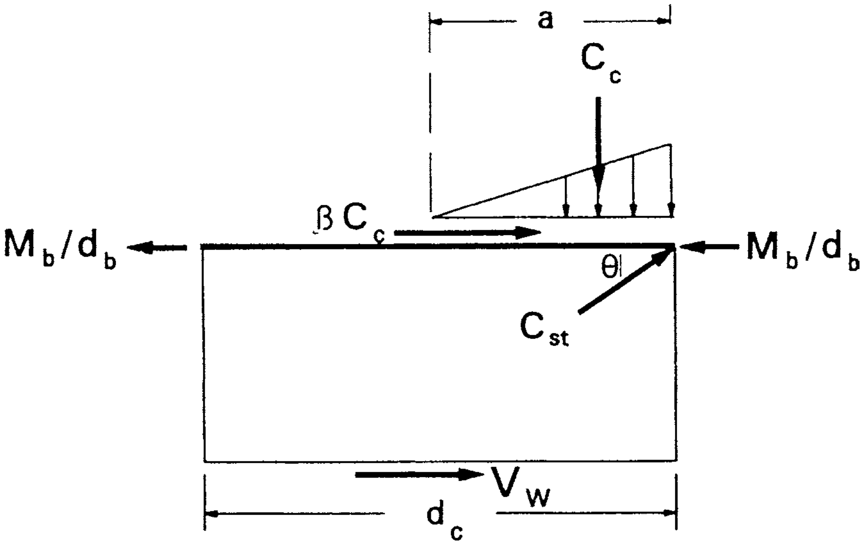

Figure 10.59 FBD of portion of web within joint area.

10.5.6.1 Design approach. Before designing the through-beam connection detail, additional equations will be derived to relate the shear stress in the beam web within the joint to the compressive force in the concrete compression strut and the externally applied forces.

Considering the FBD of a portion of the beam web within the joint area as shown in Fig. 10.59 and satisfying the horizontal force equilibrium, the following equation can be derived:

$$V_w + C_{st} \cos \theta + ßC_c - \frac{2M_b}{d_b} = 0 \qquad (10.54)$$

where V_w is the shear force in the beam web at the ultimate condition and $\theta = \arctan(d_b/d_c)$. Equations (10.52) to (10.54) can be used to proportion the through-beam connection detail.

Until further research is conducted, the following steps are suggested for designing the through-beam connection detail following the LRFD format:

1. From analysis, obtain the factored joint forces.
2. Select b_f, d_c, and f_{yl}.
3. Solving Eq. (10.53), obtain a, the depth of the neutral axis.
4. Solving Eq. (10.52), obtain t_l, the required thickness of the pipe steel.

5. Check stress in different connection elements.
6. From the vertical equilibrium requirement of the FBD shown in Fig. 10.59:

$$C_{st} = \frac{C_c}{\sin \theta} \tag{10.55}$$

Using Eq. (10.49), calculate C_c and then using Eq. (10.55) calculate C_{st}.
7. Using Eq. (10.54) calculate V_w, the shear force in the beam at the ultimate condition and compare it to V_{wy}, the shear yield capacity of the beam web given by

$$V_{wy} = 0.6F_{yw}t_w d_c \tag{10.56}$$

where F_{yw} is the beam web yield stress and t_w is the thickness of the beam web. If necessary increase the thickness of the web within the joint region. In this design procedure the assumption is that at the factored load level, the web starts to yield.
8. Check the shear stress in the concrete in the joint area. The limiting shear force could be assumed to be as suggested by ACI 352:

$$V_u = \phi R\sqrt{f_c'}A_e \tag{10.57}$$

where $\phi = 0.85$
R = 20, 15, and 12 for interior, exterior, and corner joints, respectively
f_c' = concrete compressive strength

It is suggested that the value of f_c' be limited to 70 MPa, implying that in the case of 100-MPa concrete, for instance, f_c' be taken as 70 MPa rather than 100 MPa.

10.5.6.2 Design example. Design a through-beam connection detail with the following geometry and properties:

Given steps 1 and 2:

$$b_f = 139.7 \text{ mm}$$
$$d_b = 368.3 \text{ mm}$$
$$d_c = 406.4 \text{ mm}$$
$$f_{yl} = 248.22 \text{ MPa}$$
$$F_{yw} = 248.22 \text{ MPa}$$
$$t_w = 6.35 \text{ mm}$$

$$\alpha = 0.85$$
$$l_2 = 812.8 \text{ mm}$$
$$V_b = 351.39 \text{ kN}$$
$$M_b = 187.54 \text{ kN} \cdot \text{ m}$$
$$ß = 0.5$$
$$\xi = 0.75$$
$$\eta = 4.3$$
$$f_c' = 96.53 \text{ MPa}$$
$$E_s = 200 \text{ GPa (modulus of elasticity of steel)}$$
$$E_c = 46 \text{ GPa (modulus of elasticity of concrete)}$$

Step 3: Using Eq. (10.53), calculate a, the depth of the neutral axis. Equation (10.53) will result in a third-degree polynomial which can be shown to have only one positive, real root. For this example Eq. (10.53) results in $a = 149.35$ mm.

Step 4: Using Eq. (10.52), calculate the required thickness of the steel pipe (use $t_l = 12.0$ mm):

$$t_l = \frac{149.35^2}{406.4 - 2(149.35)} \frac{1}{2(2)(4.3)} = 12.04 \text{ mm}$$

Step 5: Check stresses in different connection elements against their limit values. First calculate the tensile strain in the steel tube

$$\varepsilon_l = \frac{\xi f_{yl}}{E_s} = \frac{0.75(248.22)}{200{,}000} = 0.000931 \text{ mm/mm}$$

Using Eqs. (10.45) and (10.46), calculate f_c

$$f_c = 24.90 \text{ MPa} < f_c' = 96.53 \text{ MPa}$$

Using Eqs. (10.47) and (10.48), calculate the stresses in the other connection elements. This yields

$$f_{lc} = 108.1 \text{ MPa} < \phi_c F_y = 0.85 \times 248.22 = 211 \text{ MPa}$$
$$f_{lt} = 186.2 \text{ MPa} < \phi_t F_y = 0.9 \times 248.22 = 223.4 \text{ MPa}$$

Step 6: Using Eqs. (10.49) and (10.55), calculate the compressive force in the concrete compression strut

$$\theta = \arctan \frac{368.3}{406.4} = 42.2°$$

$$C_c = \left(\frac{1}{2}\right)(1/\eta)\zeta b_f\left(\frac{a^2}{d_c - a}\right)f_{yl}$$

$$C_c = \frac{½(0.23)(0.75)(139.7)(149.35^2)/(406.4-149.35) \times 248.22}{1000}$$

$$= 262.42 \text{ kN}$$

$$C_{st} = \frac{C_c}{\sin\theta} = \frac{262.42}{\sin 42.2} = 390.67 \text{ kN}$$

Step 7: Using Eq. (10.54), compute V_w

$$V_w + C_{st}\cos\theta + ßC_c - \frac{2M_b}{d_b} = 0$$

$$V_w + 390.67\cos(42.2) + 0.5(262.42) - \frac{2 \times 187.54 \times 10^3}{368.3} = 0$$

$$V_w = 597.79 \text{ kN}$$

From Eq. (10.56) the shear yield capacity of the beam is

$$V_{wy} = \frac{0.6 \times 248.22 \times 6.35 \times 406.4}{1000} = 384.3 \text{ kN} < 597.79 \text{ kN}$$

Since the shear yield capacity of the web within the joint is not sufficient, using Eq. (10.56), increase the web thickness to

$$t_w = \frac{597.79}{0.6 \times 248.22 \times 406.4/1000} = 9.88 \text{ mm}$$

$$t_w = 10 \text{ mm}$$

Step 8: The shear force carried by concrete within the joint between the beam flanges is assumed to be the horizontal component, C_{st}

$$V_c = C_{st}\cos\theta$$
$$V_c = 390.67\cos(42.2) = 289.41 \text{ kN}$$

For the interior joint the shear capacity is

$$V_u = \phi(20)\sqrt{f_c}(2b_f)(d_c)$$

$$V_u = 0.85(20)6.895 \times 10^{-3} \times 100 \times \frac{(2 \times 139.7)(406.4)}{1000}$$

$$= 1330.95 \text{ kN} > 289.41 \text{ kN}$$

10.6 References

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10.7 Notations (for Sec. 10.3)

a	shear span taken as one-half of coupling beam, in
A_b	cross-sectional area of stud
A_v	total area of web reinforcement in concrete encasement around steel coupling beam
A_{vd}	total area of reinforcement in each group of diagonal bars
b	width of embedded steel plate
b_f	steel coupling beam flange width
b_w	web width of encasing element around steel coupling beam
d	distance from the extreme compression fiber to centroid of longitudinal tension reinforcement in the encasing element around steel coupling beam
d'	distance from the extreme compression fiber to centroid of longitudinal compression reinforcement
d_e	distance from the stud axis to the edge of wall
d_{e1} and d_{e2}	distance from the axis of extreme studs to the edge of wall
d_h	diameter of stud head
e	eccentricity of gravity shear measured from centerline of bolts to face of wall
f_c'	concrete compressive strength, psi [for Eq. (10.2) this is for the concrete used in wall piers]

f_y	yield stress of reinforcing bars or studs
F_y	yield strength of web
h	steel coupling beam depth/distance from the center of resistance of tension studs to edge of the embedded steel plate
H	overall depth of coupling beam
I_g	gross concrete section moment of inertia
k_d	depth of concrete compression block
l_e	embedment length of studs (stud length—thickness of head)
l_n	clear span of coupling beam measured from face of wall piers
L	distance between centerlines of wall piers
L_b	clear distance between wall piers
L_e	embedment length of steel coupling beams inside wall piers
M_u	ultimate coupling beam moment
n	number of studs
P_c	tensile strength of studs based on concrete
P_s	tensile strength of studs based on steel
R	code-specified response modification factor
s	spacing of web reinforcement in encasing element around steel coupling beam
t_f	flange thickness
t_w	web thickness
t_{wall}	wall thickness, in
T_{capacity}	tensile capacity of studs
T_u	calculated tensile force in collector/outrigger beam
V_P	plastic shear capacity
V_s	shear strength of stud governed by steel
V_u	ultimate coupling beam shear force/calculated gravity shear in outrigger or collector beam
x	horizontal distance between outermost studs
y	vertical distance between studs
α	angle between diagonal reinforcement and longitudinal steel
β_1	ratio of the average concrete compressive strength to the maximum stress as defined by ACI Building Code
γ_p	coupling beam plastic shear angle
θ_e	elastic interstory drift angle
θ_p	plastic interstory drift angle
ϕ	strength reduction factor taken as 0.85