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Design of Latticed Steel Transmission Structures

This document uses both the International System of Units (SI) and customary units

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CONTENTS

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PREFACE

The first edition of this Standard was published in 1992. The second edition was published in 2000, at which time minor changes were implemented and an annex of design examples was added. From a historical perspective, the predecessor to this Standard was the ASCE-published *Guide for Design of Steel Transmission Towers*, Manual of Practice No. 52, published in 1971 and 1988. This edition of the Standard reflects minor changes to the design requirements and new sections on redundant members, welded angles, anchor bolts with base plates on leveling nuts, and post angle member splices. An example was added on how to determine the design parameter *j*. Appendix C, Guidelines for Existing Towers, was added to provide guidance when evaluating existing transmission towers.

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DESIGN OF STEEL TRANSMISSION TOWERS STANDARDS COMMITTEE

This Standard was prepared through the consensus standards process by balloting in compliance with procedures of the ASCE Codes and Standards Committee. Those individuals who serve on the Design of Steel Transmission Towers Standards Committee are

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CHAPTER 1
GENERAL

1.1 SCOPE

Design of Latticed Steel Transmission Structures specifies requirements for the design, fabrication, and testing of members and connections for electrical transmission structures. These requirements are applicable to hot-rolled and cold-formed steel shapes. Structure components (members, connections, guys) are selected to resist design-factored loads at stresses approaching yielding, buckling, fracture, or any other limiting condition specified in this Standard.

1.2 APPLICABLE DOCUMENTS

The following standards are referred to in the body of this document:

American Society for Testing and Materials (ASTM) Standards:

A6/A6M-14 *Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling*

A36/A36M-14 *Standard Specification for Carbon Structural Steel*

A123/A123-13 *Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products*

A143/A143M-07 *Standard Practice for Safeguarding Against Embrittlement of Hot-Dip Galvanized Structural Steel Products and Procedure for Detecting Embrittlement*

A153/A153M-09 *Standard Specification for Zinc Coating (Hot-Dip) on Iron and Steel Hardware*

A242/A242M-13 *Standard Specification for High-Strength Low-Alloy Structural Steel*

A394-08e1 *Standard Specification for Steel Transmission Tower Bolts, Zinc-Coated and Bare*

A529/A529M-14 *Standard Specification for High-Strength Carbon-Manganese Steel of Structural Quality*

A563-07a *Standard Specification for Carbon and Alloy Steel Nuts*

A568/A568M-14 *Standard Specification for Steel, Sheet, Carbon, Structural, and High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, General Requirements for*

A572/A572M-13a *Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel*

A588/A588M-10 *Standard Specification for High-Strength Low-Alloy Structural Steel, up to 50 ksi [345 MPa] Minimum Yield Point, with Atmospheric Corrosion Resistance*

A606/A606M-09a *Standard Specification for Steel, Sheet and Strip, High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance*

A1008/A1008M-13 *Standard Specification for Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy with Improved Formability, Solution Hardened, and Bake Hardenable*

A1011/A1011M-14 *Standard Specification for Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy with Improved Formability, and Ultra-High Strength*

C33/C33M-13 *Standard Specification for Concrete Aggregates*

American Welding Society Standard:

AWS D1.1/D1.1M: *Structural Welding Code—Steel*

1.3 DEFINITIONS

Block shear: A combination of shear and tensile failure through the end connection of a member caused by high bolt forces acting on the material; also called rupture.

Deformed bars: Steel bars meeting the requirements of ACI 318 (1983) for reinforcing bars.

Design-factored load: Unfactored load multiplied by a specified load factor to establish the design load on a structure.

Downthrust: The downward vertical component of the loads on a foundation.

Engineer of record (EOR): Prime design professional, engineering firm, or organization that is legally responsible for the tower design.

Leg member: A primary member that serves as the main corner support member of a structure; sometimes called a post member.

Line security: Criteria established to prevent a progressive (cascade) failure of structures.

Load factor: A multiplier used with the assumed loading condition, or unfactored load, to establish the design-factored load.

Primary members: Tension or compression members that carry the loads on the structure to the foundation.

Redundant members: Members that reduce the unbraced length of primary members by providing intermediate support.

Shear friction: For anchor bolts with the base assembly resting on concrete, shear is usually transferred from the base assembly to the concrete through bearing of the bolt at the surface forming a concrete wedge; translation of the wedge under the shear force cannot occur without an upward thrust of the wedge on the base assembly; this thrust induces a clamping force, and this mechanism is called shear friction.

Snug-tight: The tightness that exists when all plies in a joint are in firm contact. This tightness may be attained by a few impacts of an impact wrench or the full effort of a worker using an ordinary spud wrench. The connecting plies must be solidly seated against each other but not necessarily in continuous contact.

Tension-only member: Member with L/r greater than 300, which is assumed to be unable to resist compression.

Test engineer: The individual who has charge over the physical testing of the prototype structure.

Unfactored load: Load on a structure caused by an assumed loading condition on the wires and/or the structure; the assumed loading condition is usually identified by a combination of wind and/or ice and a temperature condition.

Uplift: The upward vertical component of the loads on a foundation.

CHAPTER 2
LOADING, GEOMETRY, AND ANALYSIS

2.1 INTRODUCTION

This Standard applies to latticed steel transmission structures. These structures shall be either self-supporting or guyed. They consist of hot-rolled or cold-formed prismatic members connected by bolts. Structure components (members, connections, guys) are selected to resist design-factored loads at stresses approaching failure in yielding, buckling, fracture, or any other specified limiting condition.

2.2 LOADS

Design-factored loads shall be determined by the purchaser and shown in the job specification either as load trees or in tabular form. These design loads shall consider the following:

1. minimum legislated levels;
2. expected climatic conditions;
3. line security provisions; and
4. construction and maintenance operations.

2.3 GEOMETRIC CONFIGURATIONS

Latticed steel structures shall be designed with geometric configurations based on electrical and safety requirements.

2.4 METHODS OF ANALYSIS

Member forces caused by the design-factored loads shall be determined by established principles of structural analysis.

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CHAPTER 3
DESIGN OF MEMBERS

3.1 INTRODUCTION

The provisions of this section are intended to apply to the design of hot-rolled and cold-formed members.

3.2 MATERIAL

Material conforming to the following standard specifications is suitable for use under this Standard:

- ASTM A36, Structural steel;
- ASTM A242, High-strength low-alloy structural steel;
- ASTM A529, Structural steel with 42,000 psi minimum yield point;
- ASTM A572, High-strength low-alloy structural columbium-vanadium steels of structural quality;
- ASTM A588, High-strength low-alloy structural steel with 50,000 psi minimum yield point to 4-in. thick, and;
- ASTM A606, Steel, sheet and strip, hot-rolled and cold-rolled, high-strength, low-alloy, with improved atmospheric corrosion resistance.

This listing of suitable steels does not exclude the use of other steels that conform to the chemical and mechanical properties of one of the listed specifications or other published specifications that establish the properties and suitability of the material.

3.3 MINIMUM SIZES

Minimum thicknesses shall be 1/8 in. (3 mm) for members and 3/16 in. (5 mm) for connection plates. See Section 7.3 for steel exposed to corrosion at the ground line.

3.4 SLENDERNESS RATIOS

Limiting slenderness ratios for members carrying calculated compressive stress shall be the following: for leg members: $L/r \le 150$; for other members: $KL/r \le 200$. The slenderness ratio KL/r for redundant members shall not exceed 250. The slenderness ratio L/r for tension-only members detailed with draw shall be greater than 300 but less than or equal to 500.

3.5 PROPERTIES OF SECTIONS

Section properties, such as area, moment of inertia, radius of gyration, section modulus, and the like, shall be based on the gross cross section, except where a reduced cross section or a net cross section is specified. The reduced cross section shall consist of all fully effective elements plus those whose widths must be considered reduced in accordance with Section 3.9.3. If all elements are fully effective, the reduced cross section and the gross cross section are identical. Net cross section is defined in Section 3.10.1.

Typical cross sections are shown in Fig. C3-1 of the Commentary of Chapter 3. The x- and y-axes are principal axes for all cross sections shown except the angle, for which the principle axes are u and z, with u being the axis of symmetry for equal leg angles.

Fig. 3-1(a) shows the method of determining w/t, the ratio of flat width to thickness of a member element. For hot-rolled sections, w is the distance from the edge of the fillet to the extreme fiber, whereas for cold-formed members it is the distance shown in Fig. 3-1(b). A larger bend radius can be used in fabrication, but for design purposes w shall be based on a maximum inside-bend radius of two times the element thickness.

3.6 DESIGN COMPRESSION

The design compressive stress F_a on the gross cross-sectional area, or on the reduced area where specified, of axially loaded compression members shall be the following:

$$F_a = \left[1 - \frac{1}{2}\left(\frac{KL/r}{C_c}\right)^2\right] F_y; \quad \frac{KL}{r} \le C_c \tag{3.6-1}$$

$$F_a = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}; \quad \frac{KL}{r} > C_c \tag{3.6-2}$$

$$C_c = \pi\sqrt{\frac{2E}{F_y}} \tag{3.6-3}$$

where

F_y = minimum guaranteed yield stress;
E = modulus of elasticity;
L = unbraced length;
r = radius of gyration;
K = effective length coefficient; and
C_c = column slenderness ratio separating elastic and inelastic buckling.

3.7 COMPRESSION MEMBERS: ANGLES

The provisions of this section are applicable only for 90° angles. If the angle legs are closed, as in a 60° angle, the provisions of Section 3.9 shall be followed.

3.7.1 Maximum *w/t* Ratio The ratio w/t, where w = flat width and t = thickness of leg, shall not exceed 25; see Fig. 3-1.

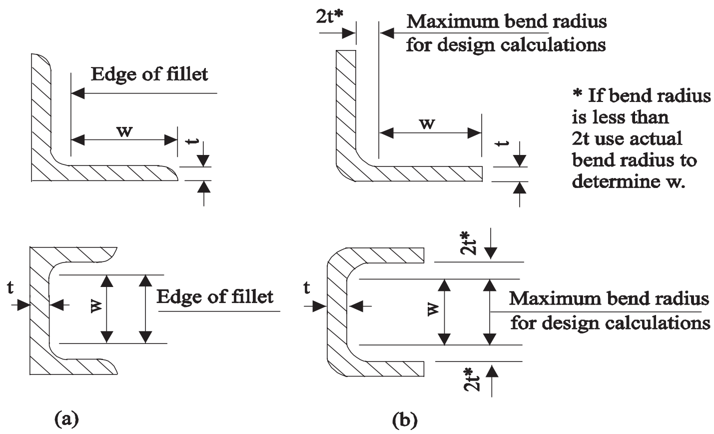


FIG. 3-1. Determination of *w/t* Ratios

3.7.2 Design Compressive Stress The design compressive stress on the gross cross-sectional area shall be the value of F_a according to Section 3.6, provided that the largest value of w/t does not exceed the limiting value given by Eq. (3.7-1).

3.7.3 Determination of F_a If w/t as defined in Section 3.7.1 exceeds $(w/t)_{\lim}$ given by

$$\left(\frac{w}{t}\right)\lim = \frac{80\Psi}{\sqrt{F_y}} \tag{3.7-1}$$

the design stress F_a shall be the value according to Section 3.6 with F_y in Eqs. (3.6-1) and (3.6-3) replaced with F_{cr} given by

$$F_{cr} = \left[1.677 - 0.677\frac{w/t}{(w/t)_{\lim}}\right]F_y; \quad \left(\frac{w}{t}\right)_{\lim} \leq \frac{w}{t} \leq \frac{144\Psi}{\sqrt{F_y}} \tag{3.7-2}$$

$$F_{cr} = \frac{0.0332\pi^2 E}{(w/t)^2} \quad \frac{w}{t} > \frac{144\Psi}{\sqrt{F_y}} \tag{3.7-3}$$

For Eqs. (3.7-1) through (3.7-3), $\Psi = 1$ for F_y in ksi and 2.62 for F_y in MPa.

3.7.4 Effective Lengths The effective length KL of members shall be determined in the following sections.

3.7.4.1 Leg Members For leg members bolted in both faces at connections,

$$\frac{KL}{r} = \frac{L}{r}; \quad 0 \leq \frac{L}{r} \leq 150 \tag{3.7-4}$$

For leg members of equal leg angles, having no change in member load between panels, used with staggered bracing, the controlling L/r values shall be as shown in Fig. 3-2.

3.7.4.2 Other Compression Members For members with a concentric load at both ends of the unsupported panel,

$$\frac{KL}{r} = \frac{L}{r}; \quad 0 \leq \frac{L}{r} \leq 120 \tag{3.7-5}$$

For members with a concentric load at one end and normal framing eccentricity at the other end of the unsupported panel,

$$\frac{KL}{r} = 30 + 0.75\frac{L}{r}; \quad 0 \leq \frac{L}{r} \leq 120 \tag{3.7-6}$$

For members with normal framing eccentricities at both ends of the unsupported panel,

$$\frac{KL}{r} = 60 + 0.5\frac{L}{r}; \quad 0 \leq \frac{L}{r} \leq 120 \tag{3.7-7}$$

For members unrestrained against rotation at both ends of the unsupported panel,

$$\frac{KL}{r} = \frac{L}{r}; \quad 120 \leq \frac{L}{r} \leq 200 \tag{3.7-8}$$

For members partially restrained against rotation at one end of the unsupported panel,

$$\frac{KL}{r} = 28.6 + 0.762\frac{L}{r}; \quad 120 \leq \frac{L}{r} \leq 225 \tag{3.7-9}$$

For members partially restrained against rotation at both ends of the unsupported panel,

$$\frac{KL}{r} = 46.2 + 0.615\frac{L}{r}; \quad 120 \leq \frac{L}{r} \leq 250 \tag{3.7-10}$$

3.7.4.3 Redundant Members For members with a concentric load at both ends of the unsupported panel, use Eq. (3.7-5).

For members with a concentric load at one end and normal framing eccentricity at the other end of the unsupported panel, use Eq. (3.7-6).

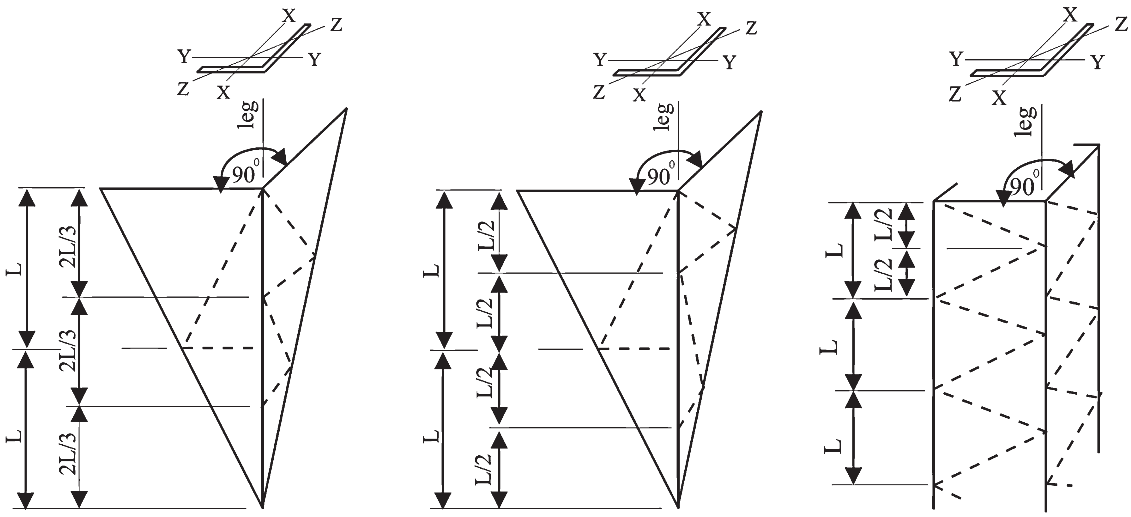
For members with normal framing eccentricities at both ends of the unsupported panel, use Eq. (3.7-7).

If members are unrestrained against rotation at both ends of the unsupported panel,

$$\frac{KL}{r} = \frac{L}{r}; \quad 120 \leq \frac{L}{r} \leq 250 \tag{3.7-11}$$

If members are partially restrained against rotation at one end of the unsupported panel,

$$\frac{KL}{r} = 28.6 + 0.762\frac{L}{r}; \quad 120 \leq \frac{L}{r} \leq 290 \tag{3.7-12}$$



(a) Leg Controlled by $(2/3L)/r_z$

Leg members shall be supported in both faces at the same elevation every four panels

(b) Leg Controlled by $(1.2L)/r_x$

Leg members shall be supported in both faces at the same elevation every four panels

(c) Leg Controlled by $(1.2L)/r_x$

For these configurations, some rolling of the leg will occur. Eccentricities at leg splices shall be minimized. The thicker leg sections shall be properly butt-spliced. The controlling L/r values shown above shall be used with a $K = 1$ as specified by Eq. 3.7-4

FIG. 3-2. Equal Leg Angle with Staggered Bracing

If members are partially restrained against rotation at both ends of the unsupported panel,

$$\frac{KL}{r} = 46.2 + 0.615\frac{L}{r}; \quad 120 \le \frac{L}{r} \le 330 \qquad (3.7\text{-}13)$$

3.7.4.4 Unsupported Length with Varying Forces When the magnitude of the force in a member (P_1 and P_2) changes over the unsupported length (L) of equal subpanels of a truss that is not braced normal to the plane of the truss, the effective length factor (K') for buckling perpendicular to the plane of the truss shall be determined as

$$K' = 0.75 + 0.25(P_2/P_1) \quad \text{where } P_1 > P_2 \text{ and both are compression forces} \qquad (3.7\text{-}14a)$$

or

$$K' = 0.75 - 0.25(P_2/P_1) \quad \text{where } P_1 = \text{compression force and } P_2 = \text{tension force} \qquad (3.7\text{-}14b)$$

The effect of partial joint restraint is accounted for by Eqs. (3.7-14a) and (3.7-14b). Additional adjustment may be required for eccentric loading (as a result of normal framing eccentricities) using Eqs. (3.7-6) and (3.7-7), where $KL/r = K(K'L)/r$.

When out-of-plane support is provided and the load in the member (P_1 and P_2) changes over the supported length (L), each subpanel of the member shall be designed as an individual member with due consideration of any eccentricities when selecting the value of KL/r.

3.7.4.5 Joint Restraint A single bolt connection at either the end of a member or a point of intermediate support shall not be considered as furnishing restraint against rotation. A multiple bolt connection, detailed to minimize eccentricity, shall be considered to offer partial restraint if the connection is to a member capable of resisting rotation of the joint. A multiple bolt connection to an angle leg or angle chord member, detailed to minimize eccentricity, shall not be considered to offer partial restraint if the connection is made only on a gusset plate without also being framed to the restraining member.

3.7.4.6 Test Verification Where tests and/or analysis demonstrate that specific details provide restraint different from the recommendations of this section, the values of KL/r specified in this section may be modified.

3.8 COMPRESSION MEMBERS: SYMMETRICAL LIPPED ANGLES

3.8.1 Maximum w/t Ratio The ratio w/t of the leg shall not exceed 60; see Fig. 3-1.

3.8.2 Design Compressive Stress The design compressive stress on the gross cross-sectional area shall be the value of F_a according to Section 3.6, provided the width-to-thickness ratio of the leg $w/t \le 220\Psi/\sqrt{F_y}$, where $\Psi = 1$ for F_y in ksi and 2.62 for F_y in MPa. If w/t exceeds $220\Psi/\sqrt{F_y}$, the design shall be based on a reduced area, according to Sections 3.5 and 3.9.3.1(b).

3.8.3 Equivalent Radius of Gyration The design stress defined in Section 3.8.2 shall be computed for the larger of KL/r_z and KL/r_{tf}, where r_{tf} is an equivalent radius of gyration given by

$$\frac{2}{r_{tf}^2}=\frac{1}{r_t^2}+\frac{1}{r_u^2}+\sqrt{\left(\frac{1}{r_t^2}-\frac{1}{r_u^2}\right)^2+4\left(\frac{u_o}{r_t r_u r_{ps}}\right)^2} \qquad (3.8\text{-}1)$$

where

$$r_t=\sqrt{\frac{C_w+0.04J\left(K_t L\right)^2}{I_{ps}}} \qquad (3.8\text{-}2)$$

C_w = warping constant;
J = St. Venant torsion constant;
K_t = effective length coefficient for warping restraint;
L = unbraced length of member;
r_u = radius of gyration about u-axis;
u_o = distance between shear center and centroid;
$r_{ps}=\sqrt{I_{ps}/A}$ = polar radius of gyration about shear center;
$I_{ps}=I_u+I_z+Au_o^2$ = polar moment of inertia about shear center;
I_u = moment of inertia about u-axis;
I_z = moment of inertia about z-axis; and
A = area of cross section.

Values of KL/r_z and KL/r_{tf} shall be determined as defined in accordance with Section 3.7.4, using $K_t = 1$ to compute r_t by Eq. (3.8-2).

3.8.4 Minimum Lip Depth The minimum depth d of a lip at the angle θ with the leg (Fig. 3-3) shall be determined by

$$d=\frac{2.8t}{(\sin\theta)^{2/3}}\sqrt[6]{\left(\frac{w}{t}\right)^2-\frac{4000\Psi}{F_y}}\geq\frac{4.8t}{(\sin\theta)^{2/3}} \qquad (3.8\text{-}3)$$

where

Ψ = 1 for F_y in ksi and 6.89 for F_y in MPa; and
w/t = flat width to thickness ratio of the leg.

The ratio w_s/t of the lip (shown in Fig. 3-3) shall not exceed $72\Psi/\sqrt{F_y}$; where $\Psi = 1$ for F_y in ksi and 2.62 for F_y in MPa.

3.9 COMPRESSION MEMBERS NOT COVERED IN SECTIONS 3.7 AND 3.8

3.9.1 Design Compressive Stress The design compressive stress on the gross cross-sectional area, or on the reduced area defined in Section 3.5 if w/t for any element exceeds the limit in Section 3.9.3.1 for which $b = w$, shall be the value of F_a according to Section 3.6. Radii of gyration used to determine F_a shall be computed for the gross cross section, and limiting values of w/t and the effective widths of elements defined in Section 3.9.3.1 shall be determined with $f = F_a$.

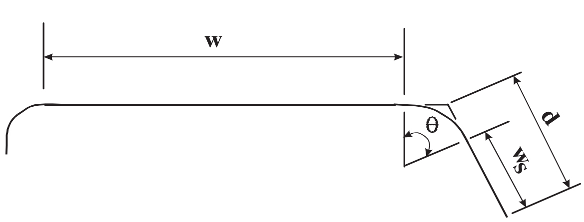


FIG. 3-3. Minimum Lip Depth

If a reduced area applies and the force P does not act at the center of gravity of the reduced area, the resulting moment shall be taken into account according to Section 3.12.

3.9.2 Maximum *w/t* Ratio The ratio w/t of flat width to thickness shall not exceed 60 for elements supported on both longitudinal edges and 25 for elements supported on only one longitudinal edge; see Fig. 3-1.

3.9.3 Effective Widths of Elements in Compression

3.9.3.1 Uniformly Compressed Elements For Eqs. (3.9-1) through (3.9-6), $\Psi = 1$ for f in ksi and 2.62 for f in MPa.

(a) The effective width b of an element supported on only one longitudinal edge shall be taken as follows:

$$b=w;\quad \frac{w}{t}\leq\frac{72\Psi}{\sqrt{f}} \qquad (3.9\text{-}1)$$

$$b=\frac{108\Psi}{\sqrt{f}}\left(1-\frac{24\Psi}{(w/t)\sqrt{f}}\right)t;\quad \frac{w}{t}>\frac{72\Psi}{\sqrt{f}} \qquad (3.9\text{-}2)$$

where f = compressive stress in an element computed for compression members as prescribed in Section 3.9.1 and for members in bending in Section 3.14.1. The effective width shall be taken adjacent to the supported edge.

(b) The effective width b of an element supported on both longitudinal edges shall be taken as follows:

$$b=w;\quad \frac{w}{t}\leq\frac{220\Psi}{\sqrt{f}} \qquad (3.9\text{-}3)$$

$$b=\frac{325\Psi}{\sqrt{f}}\left(1-\frac{71\Psi}{(w/t)\sqrt{f}}\right)t;\quad \frac{w}{t}>\frac{220\Psi}{\sqrt{f}} \qquad (3.9\text{-}4)$$

except that for flanges of square and rectangular sections,

$$b=w;\quad \frac{w}{t}\leq\frac{240\Psi}{\sqrt{f}} \qquad (3.9\text{-}5)$$

$$b=\frac{325\Psi}{\sqrt{f}}\left(1-\frac{63\Psi}{(w/t)\sqrt{f}}\right)t;\quad \frac{w}{t}>\frac{240\Psi}{\sqrt{f}} \qquad (3.9\text{-}6)$$

where f = compressive stress in an element computed for compression members as prescribed in Section 3.9.1 and for members in bending in Section 3.14.1. The portion of the element considered removed to obtain the effective width shall be taken symmetrically about the centerline.

3.9.3.2 Elements with Stress Gradient For Eqs. (3.9-7) and (3.9-8), $\Psi = 1$ for f_1 in ksi and 2.62 for f_1 in MPa.

(a) The effective width b of an element supported on only one longitudinal edge shall be determined as in Section 3.9.3.1(a), using for f the maximum compressive stress in the element.
(b) The effective widths b_1 and b_2 (Fig. 3-4) of an element supported on both longitudinal edges shall be determined as follows:

$$b_2=\frac{w}{2};\quad \frac{w}{t}\leq\frac{110C\Psi}{\sqrt{f_1}} \qquad (3.9\text{-}7)$$

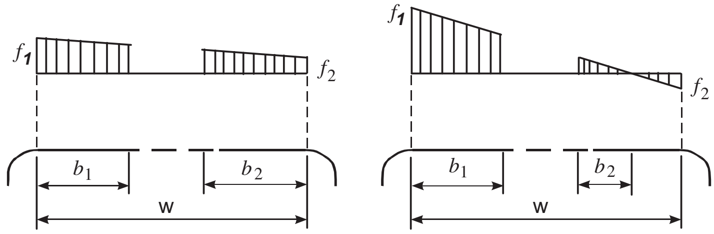


FIG. 3-4. Elements with Stress Gradient

$$b_2 = \frac{82C\Psi}{\sqrt{f_1}}\left(1 - \frac{1-36C\Psi}{(w/t)\sqrt{f_1}}\right)t; \quad \frac{w}{t} > \frac{110C\Psi}{\sqrt{f_1}} \qquad (3.9\text{-}8)$$

$$b_1 = \frac{b_2}{1.5 - 0.5\left(\frac{f_2}{f_1}\right)} \qquad (3.9\text{-}9)$$

where

$C = 2 + 0.75[1 - (f_2/f_1)]^2$;
f_1 = compressive stress shown in Fig. 3-4, to be taken positive; and
f_2 = stress shown in Fig. 3-4; positive indicates compression, negative indicates tension.

The stresses f_1 and f_2 shall be based on the reduced section, and f_1 shall be the larger if f_2 is compressive. If the sum of the calculated values of b_1 and b_2 exceeds the compressive part of the element, the element is fully effective.

3.9.4 Doubly Symmetric Open Cross Sections Members with doubly symmetric open cross sections (such as a wide flange) whose unsupported length for torsional buckling exceeds the unsupported length for flexural buckling about the weak axis shall be checked for torsional buckling as well as for flexural buckling. The design torsional-buckling stress is the value of F_a according to Section 3.6, using the radius of gyration r_t of Eq. (3.8-2) computed for the gross cross section.

3.9.5 Singly Symmetric Open Cross Sections Members with singly symmetric open cross sections (such as 60° angle or channel) shall be checked for flexural buckling in the plane of symmetry and for torsional-flexural buckling. The design torsional-flexural buckling stress is the value of F_a according to Section 3.6, using the radius of gyration r_{tf} of Eq. (3.8-1) computed for the gross cross section.[1]

3.9.6 Point-Symmetric Open Cross Sections Members with point-symmetric open cross sections (such as cruciform) shall be checked for torsional buckling as well as flexural buckling. The design torsional-flexural buckling stress is the value of F_a according to Section 3.6, using the radius of gyration r_t of Eq. (3.8-2), computed for the gross cross section.

3.9.7 Closed Cross Sections Members with closed cross sections need to be investigated only for flexural buckling.

3.9.8 Nonsymmetric Cross Sections The design compressive stress for nonsymmetric shapes shall be determined by tests and/or analysis.

3.9.9 Lips Element lips shall be dimensioned according to Section 3.8.4.

[1]Note that r_t and r_{tf} refer to the principal axes (u, z) of angles.

3.9.10 Eccentric Connections If the centers of gravity of the member connections cannot be made coincident with the center of gravity of the member cross section, either gross or reduced as applicable, the resulting bending stresses shall be taken into account according to Section 3.12.

3.10 TENSION MEMBERS

3.10.1 Design Tensile Stress The design tensile stress F_t on concentrically loaded tension members shall be F_y on the net cross-sectional area A_n, where A_n is the gross cross-sectional area A_g (the sum of the products of the thickness and the gross width of each element as measured normal to the axis of the member) minus the loss due to holes or other openings at the section being investigated. If there is a chain of holes in a diagonal or zigzag line, the net width of an element shall be determined by deducting from the gross width the sum of the diameters of all the holes in the chain and adding for each gauge space in the chain the quantity $s^2/4g$ where s = longitudinal spacing (pitch) and g = transverse spacing (gauge) of any two consecutive holes. The critical net cross-sectional area A_n is obtained from that chain which gives the least net width.

In computing net area for tension, the diameter of a bolt hole that has been punched shall be taken as 1/16 in. (1.6 mm) greater than the nominal diameter of the hole. For bolt holes that have been drilled or subpunched and reamed, the nominal diameter of the hole may be used.

Plain and lipped angles bolted in both legs at both ends shall be considered to be concentrically loaded.

3.10.2 Angle Members The design tensile stress F_t on the net area of plain and lipped angles connected by one leg shall be $0.9F_y$. If the legs are unequal and the short leg is connected, the unconnected leg shall be considered to be the same size as the connected leg. If the centroid of the bolt pattern on the connected leg is outside the center of gravity of the angle, the connection shall be checked for rupture (also called block shear) by the following equation:

$$P = 0.60A_vF_u + A_tF_y \qquad (3.10\text{-}1)$$

where

P = design tensile force on connection;
F_y = specified minimum yield strength of the member;
F_u = specified minimum tensile strength of the member;
A_v = minimum net area in shear along a line of transmitted force, see Fig. 3-5; and
A_t = minimum net area in tension from the hole to the toe of the angle perpendicular to the line of force; see Fig. 3-5.

In computing net area for tension and net area for shear, the diameter of a bolt hole that has been punched shall be taken as

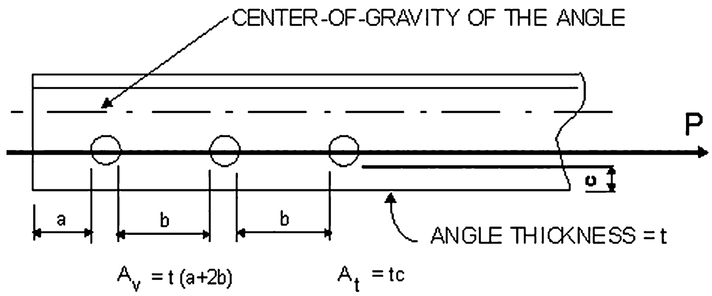


FIG. 3-5. Rupture (Block Shear) Determination

1/16 in. (1.6 mm) greater than the nominal diameter of the hole. For bolt holes that have been drilled or subpunched and reamed, the nominal diameter of the hole may be used.

3.10.3 Eccentric Connections Eccentricity of load on angle members is provided for in Section 3.10.2. Other members subjected to both axial tension and bending shall be proportioned according to Section 3.13.

3.10.4 Threaded Rods and Anchor Bolts Threaded-rod members shall have a minimum guaranteed yield F_y. The design tensile stress F_t on the stress area A_s shall be F_y. A_s is given by

$$A_s = \frac{\pi}{4}\left(d - \frac{0.974}{n}\right)^2 \qquad (3.10\text{-}2)$$

where

d = nominal diameter; and
n = number of threads per unit of length.

Anchor bolts shall have a minimum guaranteed yield F_y. See Sections 4.3.2 through 4.3.4 and Chapter 7 for design requirements.

3.10.5 Guys The design tension in guys shall not exceed 0.65 times the specified minimum breaking strength of the cable.

3.11 STITCH BOLTS

Stitch bolts shall be spaced so that the governing slenderness ratio between bolts for any component of the built-up member does not exceed the following:

For compression members: Three-quarters of the governing slenderness ratio of the built-up member.
For tension members: The governing slenderness ratio of the built-up member, or 300.

If the connected leg of a compression member exceeds 4 in. (100 mm), a minimum of two bolts shall be used at each stitch point.

3.12 AXIAL COMPRESSION AND BENDING

Eccentricity of load on angle members is provided for in Sections 3.7.4.2 and 3.8.3. Other members subjected to both axial compression and bending shall be proportioned to satisfy the following equations:

$$\frac{P}{P_a} + \frac{C_m M_x}{M_{ax}}\left[\frac{1}{1 - P/P_{ex}}\right] + \frac{C_m M_y}{M_{ay}}\left[\frac{1}{1 - P/P_{ey}}\right] \le 1 \qquad (3.12\text{-}1)$$

$$\frac{P}{P_y} + \frac{M_x}{M_{ax}} + \frac{M_y}{M_{ay}} \le 1 \qquad (3.12\text{-}2)$$

where

C_m = coefficient defined below;
P = axial compression;
P_a = design axial compression according to Section 3.6;
P_y = axial compression at yield (= F_yA);
P_{ex} = $\pi^2 EI_x/(K_xL_x)^2$;
P_{ey} = $\pi^2 EI_y/(K_yL_y)^2$;
I_x = moment of inertia about the x-axis;
I_y = moment of inertia about the y-axis;
K_xL_x, K_yL_y = effective lengths in the corresponding planes of bending;
M_x, M_y = moments about the x- and y-axes, respectively, at the point or points defined below; and
M_{ax}, M_{ay} = corresponding allowable moments according to Section 3.14 computed with C_b = 1 if Section 3.14.4 applies.

If there are transverse loads between points of support, M_x and M_y in Eq. (3.12-1) are the maximum moments *between* these points, which in Eq. (3.12-2) are the larger of the moments *at* these points. If there are no transverse loads between points of support, M_x and M_y in both Eq. (3.12-1) and Eq. (3.12-2) are the larger of the values of M_x and M_y at these points.

For restrained members with no lateral displacement of one end relative to the other, and with no transverse loads in the plane of bending between supports, $C_m = 0.6 - 0.4(M_1/M_2)$. M_1 is the smaller end moment and (M_1/M_2) is positive when bending is in reverse (S) curvature and negative when it is in single curvature. If there are transverse loads between supports, C_m = 1 for members with unrestrained ends and 0.85 if the ends are restrained.

3.13 AXIAL TENSION AND BENDING

Eccentricity of load on angle members is provided for in Section 3.10.2. Other members subjected to both axial tension and bending shall be proportioned to satisfy the following formula:

$$\frac{P}{P_a} + \frac{M_x}{M_{ax}} + \frac{M_y}{M_{ay}} \le 1 \qquad (3.13\text{-}1)$$

where

P = axial tension;
P_a = design axial tension according to Section 3.10;
M_x, M_y = the moments about the x- and y-axes, respectively; and
M_{ax}, M_{ay} = the corresponding allowable moments according to Section 3.14.

3.14 BEAMS

3.14.1 Properties of Sections Allowable bending moments shall be determined by multiplying design bending stresses F_b prescribed in the following sections by the section modulus of the gross cross section or of the reduced section defined in Section 3.5, as applicable. Radii of gyration used to determine the value of F_b for the extreme fiber in compression shall be based on the gross cross section. Effective widths of section elements shall be determined as prescribed in Section 3.9.3, using for f the stress on the element corresponding to the allowable moment defined. Limiting values of w/t shall be those given in Section 3.9.2.

3.14.2 Design Tension The design bending stress F_b on the extreme fiber in tension shall be F_y.

3.14.3 Laterally Supported Beams The design bending stress F_b on the extreme fiber in compression for members supported against lateral buckling shall be F_y.

3.14.4 I, Channel, and Cruciform Sections The design bending stress F_b on the extreme fiber in compression for doubly symmetric I sections, singly symmetric channels, and singly or doubly symmetric cruciform sections in bending about the x-axis (the x-axis is to be taken perpendicular to the web of the I and channel, but may be either principal axis for the cruciform) and not supported against lateral buckling, shall be the value of F_a according to Section 3.6 with $K = \sqrt{K_y K_t}$ and r given by

$$r^2 = \frac{C_b \sqrt{I_y}}{S_x} \sqrt{C_w + 0.04J\left(K_t L\right)^2} \qquad (3.14\text{-}1)$$

where

K_y = effective-length coefficient for y-axis bending;
K_t = effective-length coefficient for warping restraint;
I_y = moment of inertia about y-axis;
S_x = x-axis section modulus;
C_w = warping constant[2];
J = St. Venant torsion constant; and
L = unbraced length.

For members with moments M_1 and M_2 at the ends of the unbraced length,

$$C_b = 1.75 + 1.05 M_1 / M_2 + 0.3\left(M_1 / M_2\right)^2 < 2.3 \qquad (3.14\text{-}2)$$

where M_1 is the smaller end moment and M_1/M_2 is positive when bending is in reverse (S) curvature and negative when it is in single curvature.

For members for which the moment within a significant portion of the unbraced length equals or exceeds the larger of the end moments, and for unbraced cantilevers, $C_b = 1$.

The design stress on the extreme fiber in compression for the I section in bending about the y-axis shall be taken equal to F_y; for channels, see Section 3.14.7(b).

3.14.5 Other Doubly Symmetric Open Sections The design bending stress F_b on the extreme fiber in compression for laterally unsupported members of doubly symmetric open cross section not covered in Section 3.14.4 shall be the value of F_a according to Section 3.6, determined as follows.

For x-axis bending, follow Section 3.14.4 (Eq. 3.14-1); for y-axis bending, follow Section 3.14.4 (Eq. 3.14-1) but with K_x, I_x, and S_y substituted for K_y, I_y, and S_x, respectively.

3.14.6 Singly Symmetric I and T Sections The design bending stress F_b on the extreme fiber in compression for singly symmetric I-shaped members with the compression flange larger than the tension flange and for singly symmetric single-web T-shaped members with the flange in compression, in bending about the x-axis (the axis perpendicular to the web), and not supported against lateral buckling, may be taken the same as the value for a section of the same depth with a tension flange the same as the compression flange of the I or the T section. The allowable moment shall be calculated by multiplying the design stress so obtained by the compression flange section modulus of the singly symmetric shape.

The design bending stress on the extreme fiber in compression for the sections previously described, in bending about the y-axis (the axis of symmetry), shall be determined according to Section 3.14.7(a).

3.14.7 Other Singly Symmetric Open Sections The design bending stress F_b on the extreme fiber in compression for members with singly symmetric open cross section and not supported against lateral buckling, other than those covered in Sections 3.14.4 and 3.14.6, shall be the value of F_a determined according to Section 3.6 as follows:

(a) For members in bending about the axis of symmetry (the y-axis is to be taken as the axis of symmetry), use

$$K = \sqrt{K_x K_t} \text{ and } r \text{ from Eq. (3.14-1).}$$

(b) For members in bending about the x-axis (the axis perpendicular to the y-axis of symmetry), use $K = K_y$, and r given by

$$r^2 = \frac{C_b \sqrt{I_y}}{S_{xc}} \left\{ \pm j\sqrt{I_y} + \sqrt{j^2 I_y + \left(\frac{K_y}{K_t}\right)^2 \left[C_w + 0.04J\left(K_t L\right)^2\right]} \right\} \qquad (3.14\text{-}3)$$

where

$$j = \left[\frac{1}{2I_x}\int_A \left(x^2 + y^2\right) y \, dA\right] - y_o \qquad (3.14\text{-}4)$$

S_{xc} = section modulus of compression flange about x-axis;
y_o = distance from centroid to shear center;
A = area of cross section;
I_y = y-axis moment of inertia;
I_x = x-axis moment of inertia; and
K_y = effective-length coefficient for y-axis bending.
C_b, K_t, C_w, J, and L are as defined in Eqs. (3.14-1) and (3.14-2).

The positive direction of the y-axis must be taken so that the shear center coordinate y_o is negative. The plus sign for the $j\sqrt{I_y}$ in Eq. (3.14-3) is to be used if the moment causes compression on the shear center side of the x-axis, and the minus sign if it causes tension.

3.14.8 Equal Leg Angles Provided that the eccentricity of the load with respect to the shear center is not more than one-half of the leg width (Fig. 3-6), the allowable bending moment for a laterally unsupported equal leg angle shall be taken as the smaller of

(a) The moment M_{yt} that produces tensile yield stress at the extreme fiber; or

[2]For the I section, $C_w = 0.25d^2 I_y$. An approximation for the channel is $C_w = 0.15d^2 I_y$. For the cruciform section $C_w = 0$. The section depth = d.

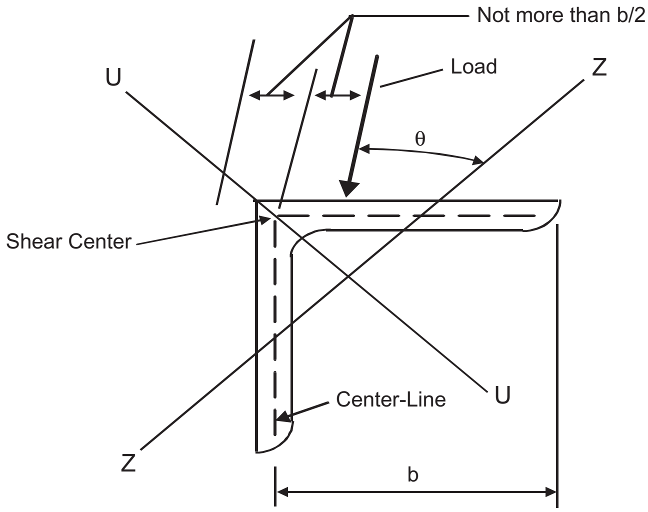


FIG. 3-6. Load on Angles

(b) The moment M_b that causes lateral buckling given by the following:

$$\text{If} \quad M_e \le 0.5M_{yc}; \quad M_b = M_e \tag{3.14-5}$$

$$\text{If} \quad M_e \ge 0.5M_{yc}; \quad M_b = M_{yc}\left(1 - \frac{M_{yc}}{4M_e}\right) \tag{3.14-6}$$

where

M_{yc} = moment causing compressive yield at extreme fiber; and
M_e = elastic critical moment.

Values of M_e are given by the following:

For load perpendicular to a leg:

$$M_e = \frac{0.66Eb^4t}{(KL)^2}\left[\sqrt{1 + \frac{0.78(KL)^2t^2}{b^4}} \pm 1\right] \tag{3.14-7}$$

For load at the angle θ with the z-axis (Fig. 3-6):

$$M_e = \frac{2.33Eb^4t}{(1 + 3\cos^2\theta)(KL)^2}\left[\sqrt{\sin^2\theta + \frac{0.156(1 + 3\cos^2\theta)(KL)^2t^2}{b^4}} \pm \sin\theta\right] \tag{3.14-8}$$

where

E = modulus of elasticity;
b = width of leg − $t/2$;
t = thickness of leg;
L = unsupported length; and
K = 1.0 if the angle is simply supported on the x- and y-axes at each end or 0.5 if it is fixed against rotation about the x- and y-axes at each end.

The plus sign for the last term (±1) in Eq. (3.14-7) and (± sin θ) in Eq. (3.14-8) applies when the load acts in the direction shown in Fig. 3-6, and the minus sign when it acts in the opposite direction.

The yield moments M_{yt} and M_{yc} are given by M_y in the following:

At the heel of the angle:

$$F_y = \pm\frac{M_y \sin\theta}{S_z} \tag{3.14-9}$$

At the toe of the angle:

$$F_y = \pm\frac{M_y \sin\theta}{S_z} + \frac{M_y \cos\theta}{S_u} \tag{3.14-10}$$

where

F_y = yield stress;
S_u and S_z = section moduli for the u- and z-axes, respectively.

The plus sign denotes tension and the minus sign compression. The applicable signs are determined according to the type of stress produced at the extreme fiber being checked. The following section moduli based on centerline dimensions may be used in lieu of those based on overall dimensions.

$$S_u = \frac{b^2t}{1.5\sqrt{2}}; \quad S_z = \frac{b^2t}{3\sqrt{2}} \tag{3.14-11}$$

3.15 DESIGN SHEAR

3.15.1 Beam Webs For Eqs. (3.15-1) through (3.15-3), Ψ = 1 for F_y in ksi and 2.62 for F_y in MPa.

The ratio h/t of the depth of a beam web to its thickness shall not exceed 200. The design average shearing stress F_v on the gross area of a beam web shall not exceed the following:

$$F_v = 0.58F_y; \quad \frac{h}{t} \le \frac{440\Psi}{\sqrt{F_y}} \tag{3.15-1}$$

$$F_v = \frac{255\sqrt{F_y}}{\Psi(h/t)}; \quad \frac{440\Psi}{\sqrt{F_y}} \le \frac{h}{t} \le \frac{557\Psi}{\sqrt{F_y}} \tag{3.15-2}$$

$$F_v = \frac{0.5\pi^2E}{(h/t)^2}; \quad \frac{h}{t} > \frac{557\Psi}{\sqrt{F_y}} \tag{3.15-3}$$

where

F_y = yield stress.

3.15.2 Angles The shear components V_1 and V_2 of the applied shear V on single-angle beams (Fig. 3-7) shall satisfy the following equations:

$$\frac{3V_1}{2b_1t} + \frac{V_2at}{J} \le 0.58F_y \tag{3.15-4}$$

$$V_2\left(\frac{3}{2b_2t} + \frac{at}{J}\right) \le 0.58F_y \tag{3.15-5}$$

where

V_1 = component of V in leg b_1;
V_2 = component of V in leg b_2;
a = distance of shear center to intersection of load plane with leg b_1;
b_1, b_2 = width of leg − $t/2$;
t = thickness of leg;
J = St. Venant torsional constant = $(b_1 + b_2)t^3/3$; and
F_y = yield stress.

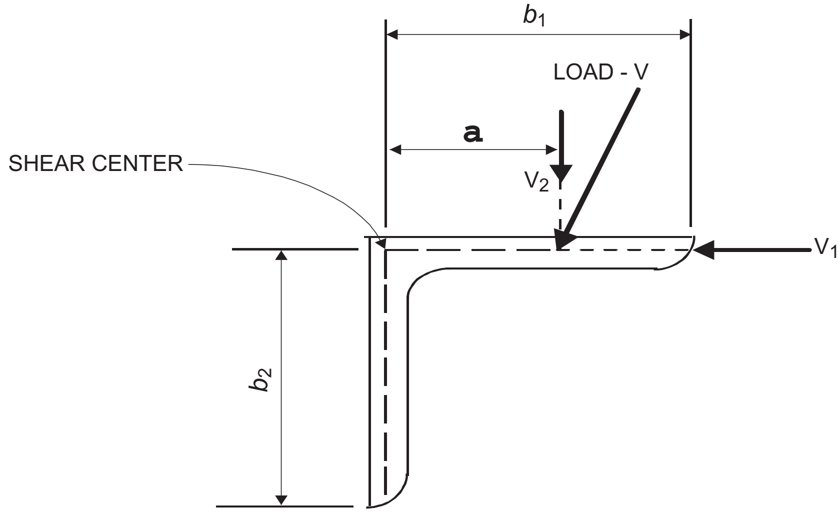


FIG. 3-7. Shear Load on Angles

3.16 REDUNDANT MEMBERS

Redundant members are used to provide intermediate bracing points to the primary members to reduce the unbraced length of these primary members.

The redundant bracing system shall be capable of resisting a force P_r, in the plane containing both primary and redundant members, as determined by

$$P_r = (0.015 + 0.008[(L/r - 60)/60])P \qquad (3.16\text{-}1)$$

where

$$0.015P \leq P_r \leq 0.025P$$

and

P = The maximum calculated compression force in the supported primary member.
L/r = The slenderness ratio of the supported primary member.

The resistance of redundant members at each node shall be combined vectorially, and the resultant resistance shall be greater than P_r. The design of redundant members shall be in accordance with Chapter 3.

3.17 WELDED ANGLES

3.17.1 Compression Members Values used for design of members in axial compression or axial compression and bending shall meet the requirements in Sections 3.7 and 3.12, respectively.

3.17.2 Tension Members Values used for design of members in axial tension or axial tension and bending shall meet the requirements in Sections 3.10 and 3.13, respectively.

3.18 TEST VERIFICATION

Design values other than those prescribed in this section may be used if substantiated by experimental or analytical investigations.

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CHAPTER 4
DESIGN OF CONNECTIONS

4.1 INTRODUCTION

Bolted connections for transmission structures are normally designed as bearing type connections. It is assumed that bolts connecting one member to another carry the load in the connection equally.

The minimum end and edge distances determined by the provisions of this chapter do not include an allowance for fabrication and rolling tolerances. Unless otherwise noted, these provisions pertain to standard holes, that is, holes nominally 1/16 in. (1.6 mm) larger than the bolt diameter.

4.2 GENERAL REQUIREMENTS

The engineer of record (EOR) shall approve the shop detail drawings; see Section 5.1.2.

4.3 FASTENERS

4.3.1 Materials The commonly used fastener specifications for latticed steel transmission towers are ASTM A394, Type 0 and 1, for bolts and A563 for nuts.

4.3.2 Bolt Shear Capacity Design shear for A394 bolts shall be the shear strength tabulated in the ASTM specification.

For bolts that do not have an ASTM-specified shear strength, the design shear stress F_v on the effective area shall be $0.62F_u$, where F_u is the specified minimum tensile strength of the bolt material. The effective area is the gross cross-sectional area of the bolt if threads are excluded from the shear plane or the root area if threads are in the shear plane.

4.3.3 Bolt Tension Capacity Bolts shall be proportioned so that the sum of the tensile stresses caused by the applied external load, and any tensile stress resulting from prying action, does not exceed the design tensile stress F_t, as follows:

(a) For bolts that have a specified proof-load stress, F_t = ASTM proof-load stress by the length-measurement method.
(b) For bolts with no specified proof-load stress, $F_t = 0.6F_u$.

The stress area A_s is given by

$$A_s = \frac{\pi}{4}\left(d - \frac{0.974}{n}\right)^2 \tag{4.3-1}$$

where

d = nominal diameter of the bolt; and
n = number of threads per unit of length.

4.3.4 Bolts Subject to Combined Shear and Tension For bolts subject to combined shear and tension, the design tensile stress $F_{t(v)}$ shall be

$$F_t(v) = F_t\left[1 - \left(f_v / F_v\right)^2\right]^{1/2} \tag{4.3-2}$$

where

F_t = design tensile stress defined in Section 4.3.3;
F_v = design shear stress defined in Section 4.3.2; and
f_v = computed shear stress on effective area.

The combined tensile and shear stresses shall be taken at the same cross section in the bolt.

4.4 DESIGN BEARING STRESS

The maximum bearing stress, calculated as the force on a bolt divided by the product of the bolt diameter times the thickness of the connected part, shall not exceed 1.5 times the specified minimum tensile strength F_u of the connected part or the bolt.

4.5 MINIMUM DISTANCES

4.5.1 End Distance (See Fig. C4-2) For stressed members, the distance e measured from the center of a hole to the end, whether this end is perpendicular or inclined to the line of force, shall not be less than the value of $e_{\min}$, determined as the largest value of e from Eqs. (4.5-1) through (4.5-3):

$$e = 1.2P/F_u t \tag{4.5-1}$$

$$e = 1.3d \tag{4.5-2}$$

$$e = t + d/2 \tag{4.5-3}$$

where

F_u = specified minimum tensile strength of the connected part;
t = thickness of the connected part;
d = nominal diameter of bolt; and
P = force transmitted by the bolt.

For redundant members, $e_{\min}$ shall be determined as the larger value of e from Eqs. (4.5-3) and (4.5-4):

$$e = 1.2d \tag{4.5-4}$$

Equation (4.5-3) does not apply for either stressed members or redundant members if the holes are drilled.

4.5.2 Center-to-Center Bolt Hole Spacing Along a line of transmitted force, the distances between centers of holes shall not be less than the value of s_{min} determined as

$$s_{min} = 1.2P/F_u t + 0.6d \qquad (4.5\text{-}5)$$

4.5.3 Edge Distance (See Fig. C4-2) The distance f from the center of a hole to the edge of the member shall not be less than the value of f_{min} given by the following.

For a rolled edge,

$$f_{min} = 0.85e_{min} \qquad (4.5\text{-}6)$$

For a sheared or mechanically guided flame-cut edge,

$$f_{min} = 0.85e_{min} + 0.0625\Psi \qquad (4.5\text{-}7)$$

where

e_{min} = end distance according to Section 4.5.1; and
Ψ = 1 for f_{min} in inches and 25.4 for f_{min} in millimeters.

4.6 ATTACHMENT HOLES

This section is valid for hole diameter to bolt diameter ratios ≤ 2. The force P for a bolt in an attachment hole shall be limited by the following:

$$P \le 0.75(L - 0.5d_h)tF_u \qquad (4.6\text{-}1)$$

or

$$P \le 1.35dtF_u \qquad (4.6\text{-}2)$$

where

L = minimum distance from the center of the hole to any member edge;
d = nominal diameter of bolt;
d_h = attachment hole diameter;
t = member thickness; and
F_u = specified minimum tensile strength of the member.

4.7 POST ANGLE MEMBER SPLICES

Butt or lap splices in post angles shall be acceptable.

4.8 TEST VERIFICATION

Design values other than those prescribed in this section may be used if substantiated by experimental or analytical investigations.

CHAPTER 5
DETAILING AND FABRICATION

5.1 DETAILING

5.1.1 Drawings Tower detail drawings consist of erection drawings, shop detail drawings, and bills of material. Erection drawings shall show the complete assembly of the structure indicating clearly the positioning of the members. Each member shall be piece-marked, and the number and lengths of bolts shall be given for each connection. Shop detail drawings shall be shown either by assembled section (in place) or piece by piece (knocked down), either hand drawn or computer drawn. Layout drawings shall be required when details are not shown by assembled sections. Computer-generated bills of material shall be acceptable unless otherwise specified by the purchaser.

5.1.2 Approval of Shop Drawings Shop detail drawings shall be approved by the engineer of record (EOR) regarding compliance with the purchaser's specifications and the strength requirements of the design. The EOR's review and approval of the shop detail drawings include responsibility for the strength of connections but do not pertain to the correctness of dimensional detail calculations, which is the responsibility of the detailer. They also do not imply approval of means, methods, techniques, sequences, or procedure of construction, or of safety precautions and programs.

5.1.3 Connections Usual detailing practice is to connect members directly to each other with minimum eccentricity. If specific joint details are required by the EOR, they shall be shown on the design drawings as referenced in the contract documents.

5.1.4 Bolt Spacing Minimum bolt spacing, and end and edge distances, as specified in the design sections of this document, shall not be underrun by mill or standard fabrication tolerances. The purchaser's specifications shall state if end distances, edge distances, and center-to-center hole spacing dimensions include provisions for mill and fabrication tolerances; if they do not, dimensions used for detailing must be adjusted to ensure that minimum dimensions are provided in the fabricated member.

5.1.5 Detail Failures During Testing If a structural failure occurs during testing of a tower, a review shall be made by the EOR to determine the reasons and to specify the required revisions.

5.1.6 Material Detail drawings shall clearly specify member and connection materials, such as ASTM specification and grade designation.

5.1.7 Weathering Steel If the structure is made of weathering steel, special detailing procedures shall be required; see Brockenbrough and Schmitt (1975) and Brockenbrough (1983).

5.1.8 Tension-Only Members Tension-only members shall be detailed sufficiently short to provide draw. Draw must consider the length and size of the member. To facilitate erection, these members shall have at least two bolts on one end. Members 15 ft (4.6 m) long, or less, are detailed 1/8 in. (3.2 mm) short. Members more than 15 ft (4.6 m) long are detailed 1/8 in. (3.2 mm) short, plus 1/16 in. (1.6 mm) for each additional 10 ft (3.1 m) or fraction thereof. If such members are spliced, the draw shall provide for the slippage at the splice.

5.1.9 Shop Check Assembly The purchaser's specifications shall include a requirement for shop assembly of new tower details, to be done partially by sections and in the horizontal position. This requirement helps validate detailing calculations and dimensions, minimize fit-up conflicts, and ensure proper assembly in the field.

5.1.10 Other Considerations All dimensions on detail drawings shall be shown with dimensional accuracy to the nearest 1/16 in. (1.6 mm).

Welded connections and built-up components shall require seal welds. Closed sections shall be detailed with vent or drain holes if they are to be galvanized. Caution shall be used to avoid explosive effects, which can injure workers or damage the component during the galvanizing process.

5.2 FABRICATION

5.2.1 Material Because various steels are used in transmission towers, a quality control program, as specified in this Standard, is necessary. All other steels shall have a special marking starting at the mill, shall be inventoried separately at the fabrication plant, and shall be properly identified during the fabricating process. Mill test reports shall be considered sufficient as certification of material, unless the purchaser's specification calls for other requirements.

5.2.2 Specifications Fabrication shall be performed according to the purchaser's specification. If this specification does not cover fabrication procedures, the latest edition of the AISC A6 specification or a specification applicable to transmission towers shall be used. These documents provide a description of acceptable fabrication methods and procedures.

5.2.3 Shop Operations Shop operations consist essentially of cutting (sawing, shearing, or flame cutting), punching, drilling, blocking or clipping, and either cold or hot bending. Hot bending requires steel to be heated to 1400–1600°F (760–871°C) if the steel is not produced to fine-grain practice; see *The Making, Shaping and Treating of Steel* (AISE 1998).

Cold bending is normally done on pieces with simple bends at small bevels. Hot bending is necessary on pieces with moderate bevels and/or compound bends; heating shall be done evenly and shall be of sufficient length and temperature to minimize necking down of the section at the bend line. Pieces requiring bends at several bevels may have to be cut, formed, and welded. Specific preparation instructions and welding symbols shall be shown on the shop detail drawings in this case.

The actual position of any punched or drilled hole on a member shall not vary more than 1/32 in. (0.8 mm) from the position for that hole shown on the shop detail drawing.

The purchaser shall review fabricators' quality control procedures and agree on methods before fabrication begins. If there is disagreement, it shall be settled in writing before fabrication.

5.2.4 Piece Marks Each tower member shall have a number conforming to the piece mark on the erection drawings stamped with a metal die. For galvanized material, these marks shall be stamped before galvanizing. Marks shall be a minimum of 1/2 in. (12.7 mm) high. For special pieces, such as anchor bolts, where die stamping is not feasible, an indelible ink marking or special tagging that is durable and waterproof shall be used. Some purchasers require that higher strength steel members include a suffix, such as "H" or "X," on the piece mark.

5.2.5 Welding This section provides criteria for welding and welded angles.

5.2.5.1 Welding Requirements Welding procedures shall comply with AWS D1.1. Special care shall be taken regarding seal welds to ensure proper galvanizing and to avoid acid "bleeding" at pockets in structural assemblies.

5.2.5.2 Welded Angles Welded angles shall only be used with the consent of the EOR.

5.2.6 Galvanizing Galvanizing shall be in accordance with ASTM A123 and A153. Procedures to avoid material embrittlement are given in ASTM A143.

5.2.7 Shipping The purchaser's specification shall clearly state the packing, bundling methods, and shipping procedures required.

CHAPTER 6
TESTING

6.1 INTRODUCTION

The purchaser shall specify in the contract documents which structures or components of structures are to be tested. If a proof test of a structure or a component of a structure is specified, the test shall be performed on a full-size prototype of the structure or component in accordance with the following sections.

6.2 FOUNDATIONS

Tests shall be performed with the prototype attached to reaction points that have the same strengths and freedoms of movement as the reaction points that will be present in the structure in service. The engineer of record (EOR) shall specify the anchorage requirements, including acceptable tolerances, in the contract documents.

6.3 MATERIAL

The prototype shall be made of material that is representative of the material that will be used in the production run. Mill test reports or coupon tests shall be available for all important members in the prototype, including, as a minimum, the members designed for only tension loading, and compression members with KL/r less than 120.

6.4 FABRICATION

Fabrication of the prototype shall be done in the same manner as for the production run.

6.5 STRAIN MEASUREMENTS

The purchaser shall specify if any special strain measurement determination methods are required for the prototype being tested.

6.6 ASSEMBLY AND ERECTION

The method of assembly of the prototype shall be specified by the purchaser. If tight bolting of subassemblies is not permitted by the construction specifications, the prototype shall be assembled and erected with all bolts finger-tight only, and tightening to final torque shall be done after all members are in place. Pickup points that are designed into the structure shall be used during erection as part of the test procedure.

6.7 TEST LOADS

The design-factored loads (see Section 2.2) shall be applied to the prototype in accordance with the load cases specified. The test specification shall state if the structure is to be tested to destruction. Wind-on-structure loads shall be applied as concentrated loads at selected points on the prototype. These loads shall be applied at panel points where stressed members intersect so that the loads can be resisted by the main structural system. The magnitudes and points of application of all loads and points of deflection measurement shall be designated by the test engineer and approved by the EOR and purchaser.

6.8 LOAD APPLICATION

Load lines shall be attached to the load points on the prototype in a manner that simulates the in-service application as closely as possible. The attachment hardware for the test shall have the same degrees of freedom as the in-service hardware.

6.9 LOADING PROCEDURE

The number and sequence of load cases tested shall be specified by the EOR and approved by the purchaser.

Loads shall be applied to 50, 75, 90, 95, and 100% of the design-factored loads. After each increment is applied, there shall be a "hold" to allow time for reading deflections and checking for signs of structural distress. The 100% load for each load case shall be held for 5 min.

Loads shall be removed completely between load cases except for noncritical load cases where, with the test engineer and/or EOR permission, the loads may be adjusted as required for the next load case. Unloading shall be controlled to avoid overstressing any members.

6.10 LOAD MEASUREMENT

Load measurement accuracy is influenced by a variety of factors. The required accuracy of the load measurements shall be designated by the test engineer and approved by the EOR and the purchaser. All applied loads shall be measured at the point of attachment to the prototype. Loads shall be measured through a verifiable arrangement of strain devices or by predetermined dead weights. Load-measuring devices shall be used in accordance with manufacturers' recommendations and calibrated before and after the conclusion of testing.

6.11 DEFLECTIONS

Deflection measurement accuracy is influenced by a variety of factors. The required accuracy of the deflection measurements shall be designated by the test engineer and approved by the EOR and the purchaser. Structure deflections under load shall be measured and recorded as specified by the test engineer and/or

EOR. Deflection readings shall be made for the before- and off-load conditions, as well as at all intermediate holds during loading.

All deflections shall be referenced to common base readings taken before the first test loads are applied.

6.12 FAILURES

When a premature structural failure occurs, the cause of the failure, the corrective measures to be taken, and the need for a retest shall be determined by the EOR and approved by the purchaser.

Failure of individual structural members or connections that under further load application cause load redistribution through the structure but do not cause collapse of the structure (or part of the structure) shall not be considered a structural failure.

If a retest is ordered, failed members and members affected by consequential damage shall be replaced. The load case that caused the failure shall be repeated. Load cases previously completed need not be repeated.

After completion of testing, the prototype shall be dismantled and all members shall be inspected. Permanent deformations in connection plates such as load attachment points may be strengthened without the need for a retest. The following shall not be considered as failures:

(a) Residual bowing of members designed for only tension;
(b) Ovalization of no more than one-half the holes in a connection; and
(c) Slight deformation of no more than one-half the bolts in a connection.

6.13 DISPOSITION OF PROTOTYPE

The test specification shall state what use may be made of the prototype after the test is completed.

6.14 REPORT

The testing organization shall furnish the number of copies required by the job specifications of a test report that shall include the following:

(a) The designation and description of the prototype tested;
(b) The name of the purchaser;
(c) The name of the test engineer;
(d) The name of the engineer of record (EOR);
(e) The name of the fabricator;
(f) A brief description and the location of the test frame;
(g) The names and affiliations of the test witnesses;
(h) The dates of each test load case;
(i) Design and detail drawings of the prototype, including any changes made during the testing program;
(j) A rigging diagram with details of the points of attachment to the prototype;
(k) Calibration records of the load-measuring devices;
(l) A loading diagram for each load case tested;
(m) A tabulation of deflections for each load case tested;
(n) In case of failure:
Photographs of the failure;
Loads at the time of failure;
A brief description of the failure;
The remedial actions taken;
The dimensions of the failed members; and
Test coupon reports of failed members;
(o) Photographs of the overall testing arrangement and rigging;
(p) Air temperature, wind speed and direction, any precipitation, and any other pertinent meteorological data;
(q) Mill test reports as submitted according to the requirements of Section 6.3; and
(r) Additional information specified by the purchaser.

CHAPTER 7
STRUCTURAL MEMBERS AND CONNECTIONS USED IN FOUNDATIONS

7.1 INTRODUCTION

This chapter specifies design procedures for steel members and connections embedded in concrete foundations or the earth. Additional design requirements for structural members and connections of grillages, pressed plates, anchor bolts, and stub angles are covered in Chapters 3 and 4. Fig. 7-1 illustrates some typical foundations.

7.2 GENERAL CONSIDERATIONS

7.2.1 Steel Grillages Grillages shall be checked for overturning stability. The members forming the pyramid (tetrapod), Fig. 7-1(a), shall be designed considering no lateral support from the surrounding soil.

The stub angle, or leg member, Fig. 7-1(b), shall be designed considering support only at the shear plate and the base of the grillage. When the shear plates are omitted, the stub angle shall be designed for axial load and bending moment based on passive soil pressure along the length of the stub.

7.2.2 Pressed Plates The stub angle, or leg member, Fig. 7-1(c), shall be designed considering support only at the pressed plate base and the shear plate. When the shear plates are omitted, the stub angle shall be designed for axial load and bending moment based on passive soil pressure along the length of the stub.

7.2.3 Stub Angles in Concrete Piers The tensile and compressive loads in the stub angle, Fig. 7-1(d), shall be transferred to the concrete by the bottom plate or the shear connectors shown in Fig. 7-3. The shear load shall be transferred to the concrete by side bearing pressure.

7.2.4 Anchor Bolts [See Fig. 7-1(e)]

7.2.4.1 Smooth Bars with Base Assembly in Contact with Concrete or Grout The anchor bolt shall be designed to transfer the tensile load to the concrete by the end connection.

The compressive load shall be transferred to the concrete or grout by the base assembly. The shear load is assumed to be transferred to the concrete by shear friction based upon the clamping force on the base assembly.

7.2.4.2 Deformed Bars with Base Assembly in Contact with Concrete or Grout The anchor bolt shall be designed with sufficient embedded length to transfer the tensile load to the concrete by the bond between the bolt and the concrete. If the anchor bolt lacks sufficient embedment length, the tensile load shall be transferred to the concrete by the end connection. The compressive load shall be transferred to the concrete or grout by the base assembly. The shear load is assumed to be transferred to the concrete by shear friction based upon the clamping force on the base assembly.

7.2.4.3 Smooth or Deformed Bars with Base Assembly Not in Contact with Concrete or Grout If the base assembly is permanently supported on anchor bolt leveling nuts, the transfer of the tensile or compressive load to the concrete shall conform to the following:

(a) For smooth bars by the end connection; and
(b) For deformed bars by the bond between the concrete and the bar; if sufficient embedment length is not provided, the end connection shall take the entire load.

The shear load shall be transferred to the concrete by side bearing pressure. The anchor bolt shall be checked for a combination of tension, bending, and shear, as well as compression, bending, and shear.

7.3 DETERIORATION CONSIDERATIONS

Steel that is galvanized, or otherwise protected, shall have a minimum thickness of 3/16 in. (4.8 mm) when exposed to corrosion at the ground level or below.

7.4 DESIGN OF STUB ANGLES AND ANCHOR BOLTS

7.4.1 Stub Angles in Concrete The stub angle, at the plane of the intersection with the concrete, shall be checked for a combination of tension plus shear and compression plus shear, as follows:

$$A_a = \frac{P}{F_y} + \frac{V}{0.75F_y} \tag{7.4-1}$$

where

A_a = gross area of stub angle, or net area, if there is a hole at the intersecting plane;
P = tensile or compressive load on the stub angle;
V = shear load parallel to the intersection plane; and
F_y = specified minimum yield strength of stub angle.

7.4.2 Anchor Bolts with Base Assembly in Contact with Concrete or Grout When the anchor bolt bases are subjected to uplift and shear loads, the shear load shall be assumed to be transferred to the concrete by shear friction based upon the clamping force of the anchor bolts. The area of steel required shall be

$$A_s = \frac{T}{F_y} + \frac{V}{(\mu)0.85F_y} \tag{7.4-2}$$

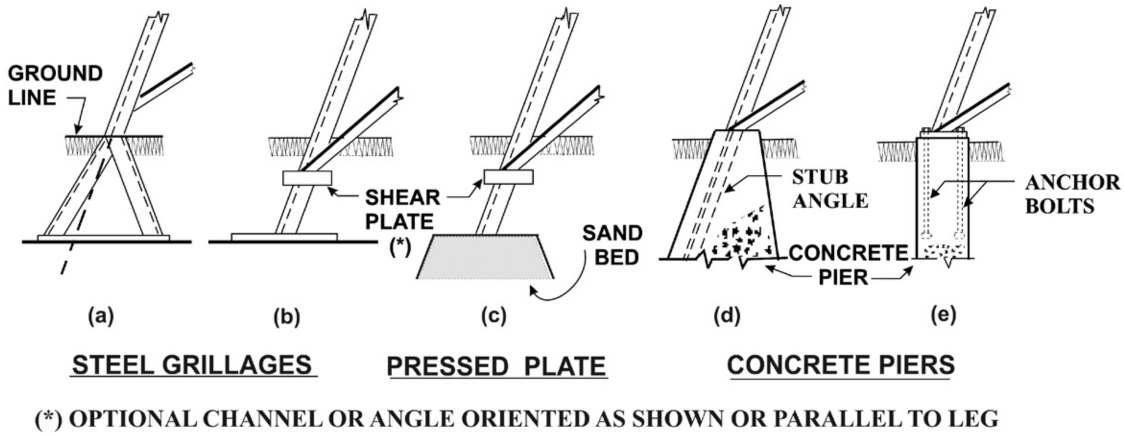

FIG. 7-1. Typical Foundations

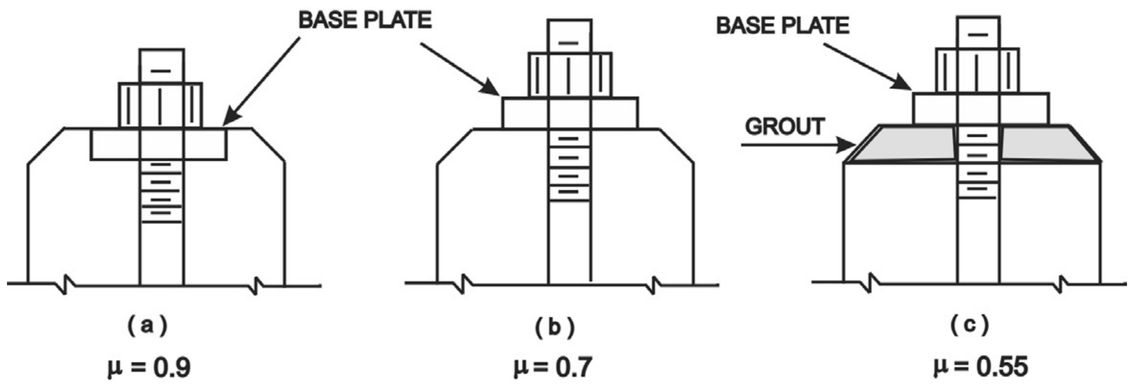

FIG. 7-2. Coefficient of Friction (μ) Values for Various Conditions

The stress area through the threads is given by the following:

$$A_s = \frac{\pi}{4}\left(d - \frac{0.974}{n}\right)^2 \tag{7.4-3}$$

where

T = tensile load on anchor bolt;
V = shear load perpendicular to anchor bolts;
F_y = specified minimum yield strength of anchor bolt;
d = nominal diameter;
n = number of threads per unit of length; and
μ = coefficient of friction.

The values for μ (Fig. 7-2) are the following:

(a) 0.9 for concrete or grout against as-rolled steel with the contact plane a full plate thickness below the concrete surface;
(b) 0.7 for concrete or grout placed against as-rolled steel with contact plane coincidental with the contact surface; and
(c) 0.55 for grouted conditions with the contact plane between grout and as-rolled steel above the concrete surface.

When anchor bolt bases are subjected to a shear load or a combination of downthrust and shear loads, the anchor bolt area shall be checked by

$$A_s = \frac{V - 0.3D}{(\mu)0.85F_y} \tag{7.4-4}$$

where D = downthrust load, and the other terms are as defined after Eq. (7.4-3).

When shear lugs are attached to the base assembly to transfer the shear to the concrete, the area of the anchor bolt need not be checked by Eqs. (7.4-2) and (7.4-4).

A combination of shear lugs and shear friction is not allowed.

7.4.3 Anchor Bolts with Base Plates on Leveling Nuts Anchor bolts on leveling nuts shall be designed according to the criteria in ASCE 48 (2011), Section 9.3.3.

7.5 DESIGN REQUIREMENTS FOR CONCRETE AND REINFORCING STEEL

The ultimate design stresses and strength factors of ACI 318 (2014) shall be used for the design of concrete and reinforcing steel in conjunction with the structure design-factored loads specified in Chapter 2.

7.5.1 Stub Angles When a bottom plate is used [Fig. 7-3(a)], the plate shall transfer the entire load in the stub angle to the concrete; concrete anchorage value shall be determined by the requirements of Section 7.5.2.

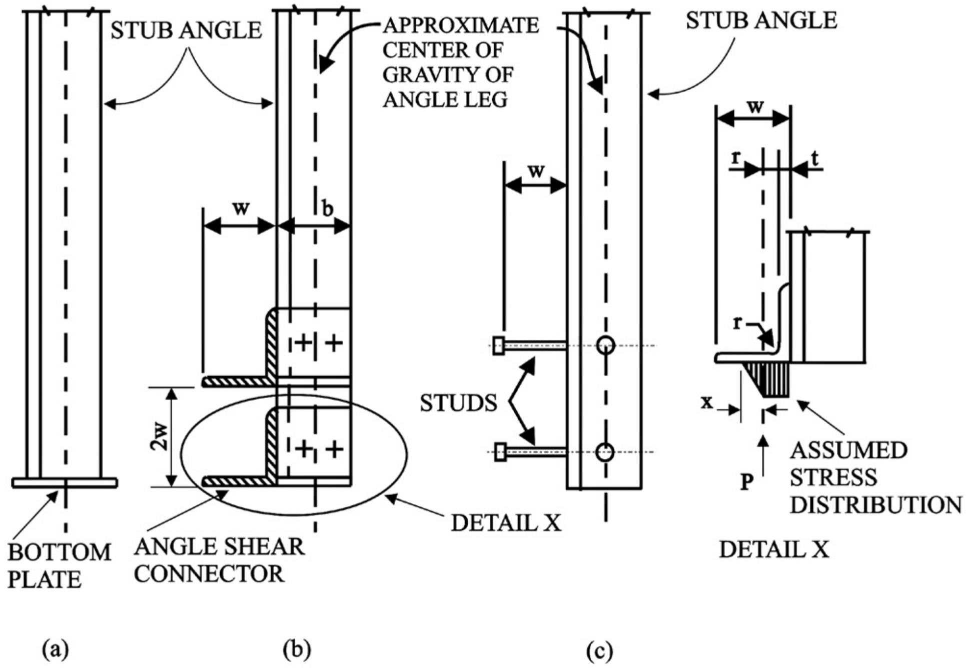

FIG. 7-3. Stub Angles

When shear connectors [Fig. 7-3(b) or 7-3(c)] are used and spaced along the length of the stub angle, the requirements of Section 7.6 shall apply.

7.5.2 Smooth Bar Anchor Bolts The anchorage value shall be limited by the pull-out strength of the concrete based on a uniform tensile stress, in ksi, of $0.126\phi\sqrt{f_c'}$ (in MPa, of $0.33\phi\sqrt{f_c'}$), acting on an effective stress area that is defined by the projecting area of stress cones radiating toward the surface from the bearing edge of the anchors.

The effective area shall be limited by overlapping stress cones, by the intersection of the cones with concrete surfaces, by the bearing area of anchor heads, and by the overall thickness of the concrete. The angle for calculating projected area shall be 45°. The φ-factor shall be 0.65 for an embedded anchor head. When there is more than one anchor bolt in a line, overlapping stress cones shall be taken into account in determining the effective area.

The anchor head can be a nut, bolt head, or plate. The bearing requirements of ACI 318 (1983) need not be met if the anchor head satisfies the following conditions:

(a) The bearing area of the anchor head (excluding the area of the anchor bolt) is at least 1.5 times the area of the anchor bolt;
(b) The thickness of the anchor head is at least equal to the greatest dimension from the outermost bearing edge of the anchor head to the face of the anchor bolt; and
(c) The bearing area of the anchor head is approximately evenly distributed around the perimeter of the anchor bolt.

7.5.2.1 Minimum Embedment for Anchor Bolts The minimum embedment depth shall be $12d\sqrt{F_u / 58\Psi}$ where

d = nominal diameter;
F_u = specified minimum tensile strength; and
Ψ = 1 for F_u in ksi and 6.89 for F_u in MPa.

7.5.3 Deformed Bar Anchor Bolts The embedment for deformed bars that are threaded and used as anchor bolts shall be in accordance with ACI 318 (1983); see Sections 7.2.4.2 and 7.2.4.3. Bars Grade 60 and above shall have a minimum Charpy-V notch requirement of 15 ft-lb (20 m-N) at −20°F (−29°C), when tested in the longitudinal direction.

7.6 SHEAR CONNECTORS

7.6.1 Stud Shear Connectors [See Fig. 7-3(c)] The capacity, Q_n, of a stud shear connector shall be as given by Eq. (7.6-1), but not to exceed the value determined by Section 4.3.2.

$$Q_n = 0.5\phi A_{sc}\sqrt{f_c' E_c} \tag{7.6-1}$$

where

ϕ = 0.85;
A_{sc} = cross-sectional area of a stud shear connector;
f_c' = specified compressive strength of concrete; and
E_c = modulus of elasticity of concrete.

Values of Q_n from Eq. (7.6-1) are applicable only to concrete made with ASTM C33 aggregates. All AISC requirements (AISC 2011) for stud material and configuration, spacing, ratio of stud diameter to minimum thickness of material to which it is welded, and concrete properties and coverage shall be met.

7.6.2 Angle Shear Connectors [See Fig. 7-3(b)] The capacity, P, of angle shear connectors shall be determined by the following:

$$P = 1.19 f_c' b(t + r + x/2) \tag{7.6-2}$$

and

$$x = t\left[\frac{F_y}{1.19 f_c'}\right]^{1/2} \le w - r - t \tag{7.6-3}$$

where

f_c' = specified compressive strength of concrete;
b = length of angle shear connector;
t = thickness of angle shear connector;
r = radius of fillet;
F_y = specified minimum yield strength of steel; and
w = width of angle shear connector leg.

The angle shear connector shall be located with its length symmetrical about the center of gravity of the stub angle leg. The connector shall be fastened to the stub with sufficient bolts or welds to take both shear and moment. The minimum center-to-center spacing of shear connectors shall be $2w$.

7.7 Test Verification

Design values other than those prescribed in this section may be used if substantiated by experimental or analytical investigations.

CHAPTER 8
QUALITY ASSURANCE AND QUALITY CONTROL

8.1 INTRODUCTION

The contract between the purchaser and the supplier shall state the responsibilities of each party and the conditions under which the work will be accepted or rejected.

8.2 QUALITY ASSURANCE

Quality assurance (QA) is the responsibility of the purchaser. The purchaser's bid documents shall outline the QA methods, types of inspections and records that will be required to determine the acceptability of the product at each stage of the design, manufacturing, and construction process.

8.3 QUALITY CONTROL

Quality control (QC) is the responsibility of the supplier. The supplier shall have a QC program consisting of a written document or a series of departmental memoranda that establishes the procedures and methods of operation that affect the quality of the work.

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COMMENTARY

This Commentary is not a part of the ASCE Standard. It is included for informational purposes only. This information is provided as explanatory and supplementary material designed to assist in applying the recommended requirements.

The sections of this Commentary are numbered to correspond to the sections of the Standard to which they refer. Since it is not necessary to have supplementary material for every section in the Standard, there are gaps in the numbering sequence of the Commentary.

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CHAPTER C2
LOADING, GEOMETRY, AND ANALYSIS

C2.1 INTRODUCTION

Design-factored loads are loads multiplied by load factors. The overload capacity factors specified by the *National Electrical Safety Code* (IEEE 2012) are load factors according to the terminology of this Standard.

C2.2 LOADS

Extensive background information on the selection of design-factored loads can be found in *Guidelines for Electrical Transmission Line Structural Loading* (ASCE 2009) and the *National Electrical Safety Code* (IEEE 2012).

Minimum legislated loads are specified in applicable codes covered by state and local authorities. Local climatic conditions may dictate loads in excess of legislated loads. These conditions may include wind, ice, or both combined at a specified temperature. Line security, which covers measures to prevent progressive line failure (cascading), should be addressed. This type of security is often handled by specifying one or more longitudinal load conditions. Loads from anticipated construction and maintenance operations should be specified to ensure the safety of the personnel involved in those operations.

It is suggested that tower members that may be used for support by maintenance personnel when climbing a tower be capable of supporting a vertical load as defined in ASCE Manual of Practice 74 (2009) applied independently of all other loads without permanent distortion of the member. If end connection assembly bolts are properly tightened, the frictional restraining effect may be considered.

It is suggested that tower members used to support maintenance personnel when climbing a tower be capable of supporting a vertical worker load applied independently of all other loads without permanent distortion of the member. Recommendation for worker load is provide in ASCE Manual of Practice 74 (2009). If end connection assembly bolts are properly tightened, the frictional restraining effect may be considered.

If fall protection is required, it should be considered in the design of the tower. The purchaser should provide the engineer of record (EOR) with all climbing requirements for the structure in accordance with IEEE 1307 (2004) or OSHA specifications. The EOR should provide a tower design that meets the purchaser's requirements, including appropriate attachment provisions.

C2.3 GEOMETRIC CONFIGURATIONS

Three basic structure definitions are recommended: suspension, strain, and dead-end structures. The conductor phases pass through and are suspended from the insulator support points of a suspension structure. The strain structure conductor attachment points are made by attaching the conductor to a dead-end clamp or a compression or bolted fitting and connecting the clamp, through the insulator string, directly to the structure. A jumper is looped through or around the structure body to electrically connect the adjacent spans. Dead-end structure conductor attachments are made the same way as for the strain structure.

Dead-end structures often have different tensions or conductor sizes on opposite sides of the structure; this arrangement creates an intact unbalanced longitudinal load. Overhead ground wires are attached to the structure using similar methods as outlined for the conductors. Additional nomenclature for the basic structure types is used to help identify the line angle at a particular structure.

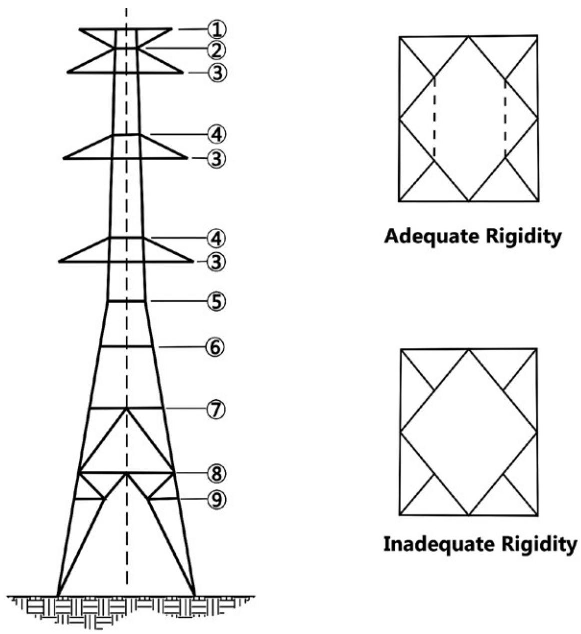
The term "tangent" is prefixed to a basic structure type for zero line angle, and the term "angle" is used when there is a line angle. Therefore, the following terminology is recommended: tangent suspension, angle suspension, tangent strain, angle strain, tangent dead-end, and angle dead-end.

Guyed structures rely on internal or external guy cables for their stability. They are normally less rigid than self-supporting structures, and their deflections may affect electrical clearances.

In most structures, horizontal bracing in the face of the tower, with an internal diaphragm, is required to provide adequate rigidity and to distribute shear and torsional forces. It is normally used at levels where there is a change in the slope of the structure leg. Horizontal bracing is also used in square and rectangular configuration structures to support horizontal struts and to provide a stiffer system to assist in reducing distortion caused by torsional and/or oblique wind loads. The spacing of horizontal bracing is dictated by general stiffness requirements to maintain tower geometry and face alignment. Factors that affect this determination are the following: type of bracing system, face slope, dead load sag of the face members, and erection considerations that affect splice locations and member lengths. In typical self-supporting towers, the recommended locations for plan bracing with or without a diaphragm are shown on Fig. C2-1. For structures that are taller than 200 ft. (61 m), or heavy dead-end towers, it is suggested that horizontal bracing with a diaphragm be installed at intervals not exceeding 75 ft. (23 m).

C2.4 METHODS OF ANALYSIS

A lattice structure is described by a one-line design drawing, which shows overall dimensions, member sizes, and locations. Because of the high degree of symmetry of most latticed structures, a transverse view, a longitudinal view, and a few horizontal cross-sectional or plan views are sufficient to describe the entire structure (Fig. C2-2). For purposes of analysis, a latticed structure is represented by a model composed of members (and



LOCATION	COMMENTS
1 Top	Diaphragm optional
2	Diaphragm recommended
3 Torsion level	Diaphragm recommended
4	Optional, unless hanger is in compression
5 Change in slope of post member	Diaphragm recommended
6 Intermediate level	It is suggested that diaphragm be installed at intervals not exceeding 75 ft (23 m). In the case when diaphragm is omitted and the horizontal member is continuous, the horizontal member can be treated as a redundant member as defined in Section 3.7.4.3. Also horizontal deflection should be considered when the tower face width is large.
7, 8 Basic tower and body extension interface	Diaphragm is optional, however Section 3.7.4.4 is applicable when it is omitted.
9 Top of leg extensions	Diaphragm (oblique face bracing) is recommended with due consideration given to oblique face stability

FIG. C2-1. Potential Diaphragm Locations

sometimes cables) interconnected at joints. Members are normally classified as primary and redundant members. Primary members form the triangulated system (three-dimensional truss) that carries the loads from their application points to the foundation. Redundant members are used to provide intermediate bracing points to the primary members to reduce the unbraced lengths of these primary members. They can easily be identified on a drawing (see the dotted lines in Fig. C2-2) as members inside triangles formed by primary members.

The locations of the joints in any model should be at the intersections of the centroidal axes of the members. Slight deviations from these locations do not significantly affect the distribution of forces.

Latticed structures are analyzed almost exclusively as ideal elastic three-dimensional trusses made up of straight members or cables and pin-connected at joints. Such elastic analyses produce only joint displacements, tension, and compression in members and tension in cables; moments from normal framing eccentricities are not calculated in the analysis. Moments in members caused by framing eccentricities, eccentric loads, distributed wind load on members, and the like can affect the member selection. These moments are considered in the member selection by the procedures described in Chapter 3.

First-order linear elastic truss analysis treats all members as linearly elastic (capable of carrying compression as well as tension) and assumes that the loaded configuration of the structure (used to verify final equilibrium) is identical to its unloaded configuration. Therefore, in a first-order linear analysis, the secondary effects of the deflected structure are ignored and the forces in the redundant members are equal to zero. The redundant members need not be included in this type of analysis, since they have no effect on the forces in the load-carrying members. This type of analysis is generally used for conventional, relatively rigid, self-supporting structures.

In a second-order (geometrically nonlinear) elastic analysis, structure displacements under loads create member forces in addition to those obtained in a first-order analysis. These additional member forces are called the $P\Delta$ effects in building

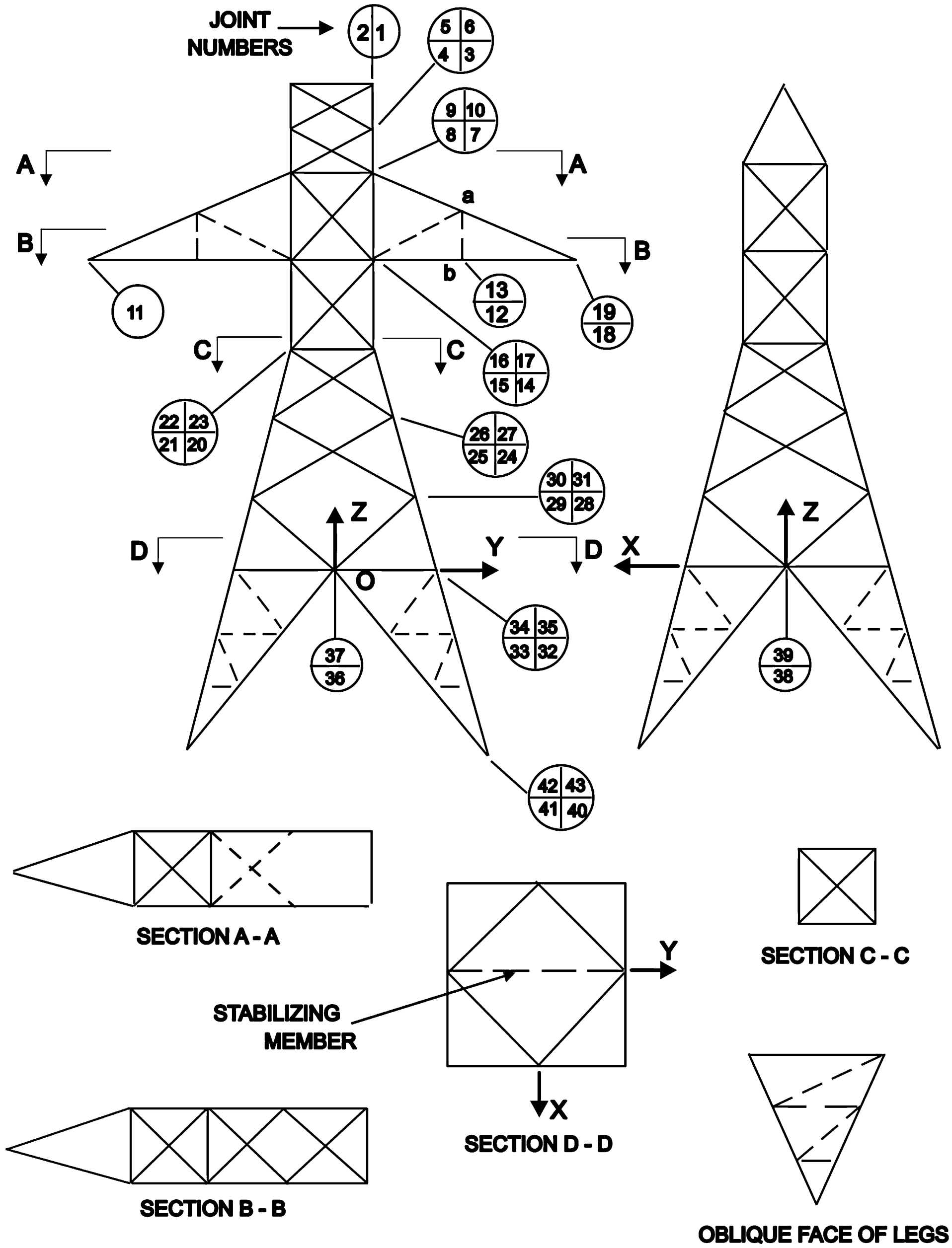


FIG. C2-2. Model of Simplified Tower

frameworks or transmission pole structures. They are automatically included in a second-order (or geometrically nonlinear) analysis that produces member forces which are in equilibrium in the deformed structure configuration (Peyrot 1985). A second-order elastic analysis may show that redundant members carry some load. Flexible self-supporting structures and guyed structures normally require a second-order analysis.

For purposes of analysis, it is sometimes assumed that bracing members with an L/r value greater than 300 cannot resist compressive loads. Such members are called tension-only members (see Section 5.1.8). Fig. C2-3 shows the difference in load distribution between a tension-compression system and a tension-only system.

Other methods of analysis, which account for the load distribution in bracing members, may be used. There is some question as to the load that can be carried by a bracing member strained in compression beyond the value e_{max} that corresponds to its theoretical capacity F_{max}, Fig. C2-4(a). Three different analysis methods have been used to model postbuckling member behavior. Method 1 assumes that the member is still capable of carrying its buckling load, irrespective of the amount of strain beyond e_{max}, Fig. C2-4(b). Method 2 assumes that the member carries no load after passing e_{max}, Fig. C2-4(c). Method 3 assumes that the compression member can carry a reduced load beyond e_{max} (Prickett et al. 1989); the member load is modeled using a postbuckling performance curve, Fig. C2-4(a).

When performing a computer analysis of an existing structure, careful attention should be given to the method of analysis used when the structure was originally designed (Kravitz 1982). If the structure was originally designed by manual (algebraic or

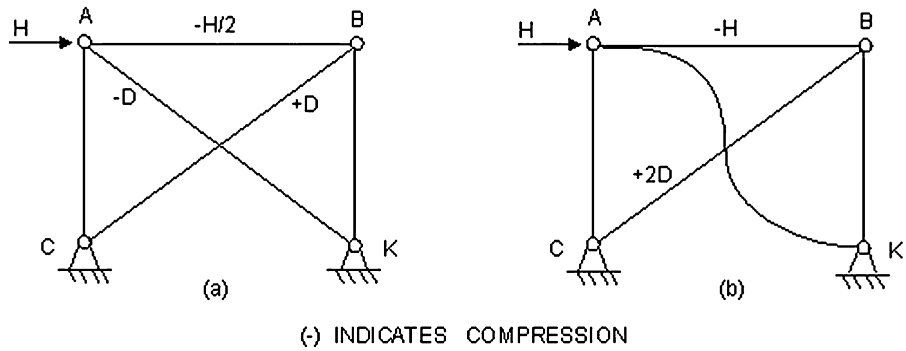


FIG. C2-3. (a) Tension-Compression System; (b) Tension-Only System

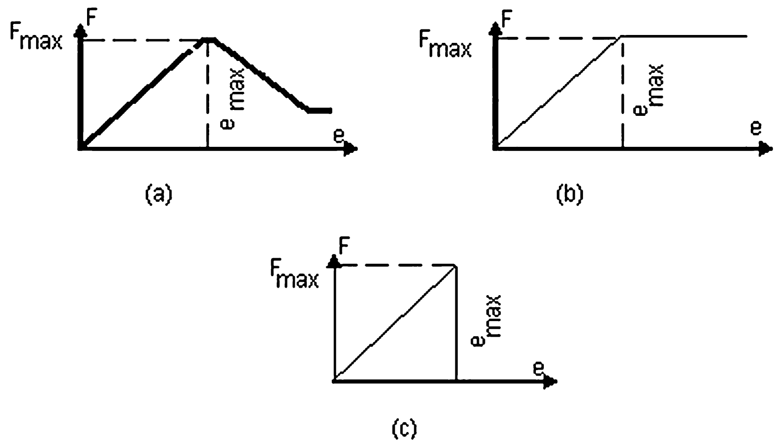


FIG. C2-4. Relationships Between Member Compression Force and Shortening: (a) Actual Member; (b) Liberal Assumption; (c) Conservative Assumption

graphical) methods and the design loads are not changed, a three-dimensional computer analysis may indicate forces in the members that are different from those from the manual methods. The EOR should determine and document why the differences exist before proceeding with the new analysis. If the tower is to be upgraded and new design-factored loads specified, then it is normally more cost-effective to rely on a computer analysis. A correlation of past model assumptions with present model assumptions should be performed for the entire structure. Detail drawings should be reviewed to ensure that members and connections are in agreement with the original design drawings.

If the included angle between a bracing member and the member it supports is small, the bracing member should not be considered as providing full support; as the included angle approaches 15º, the supported member should be investigated for stability.

Moments can occur in a leg or cross-arm chord if the bracing is insufficiently stiff (Roy et al. 1984). Fig. C2-5(a) represents qualitatively a situation where the bracing is sufficiently stiff to carry all the shear load *H*. Fig. C2-5(b) represents a case where the bracing system, because of the small value of the angle and small diagonal member sizes, is insufficiently stiff; therefore, the diagonals carry only a portion of the shear; the remainder of the shear produces moments in the vertical members *AC* and *BD*. If significant moments are anticipated in leg members, it is prudent to use an analysis method that models such members as beams. In this case, the beam members should be modeled with their section properties based on the principal axes. Also, the principal axes have to be properly oriented in the computer model to reflect the physical orientation of the angles. Other members in the structure can still be modeled as truss elements.

Dynamic analysis of latticed structures can be performed with a general-purpose finite element computer program. However, there is no indication that such an analysis is needed for design purposes, even in earthquake-prone areas (Long 1974).

Specialized computer programs for the analysis of latticed transmission structures (e.g., Rossow et al. 1975; Power Line Systems 2011; and *ATADS* 2012) should include the following features: (1) automatic generation of nodes and members that use linear interpolations and symmetries, (2) interactive graphics to ascertain model correctness, (3) provisions for tension-only members and (4) automatic handling of planar nodes, and mechanisms (unstable subassemblies) that may develop in a small group of nodes and members. Out-of-plane instabilities or mechanisms are generally prevented in actual structures by the bending stiffness of continuous members that pass through the joints.

Joints 3 through 6 of Fig. C2-2 are planar nodes, that is, all members meeting at those points lie in the same plane. Joints 12 and 13 are also planar joints if the redundant member "*ab*" is not included in the model. The horizontal bracing in section *D–D* is a mechanism in the absence of the member shown as a dotted line.

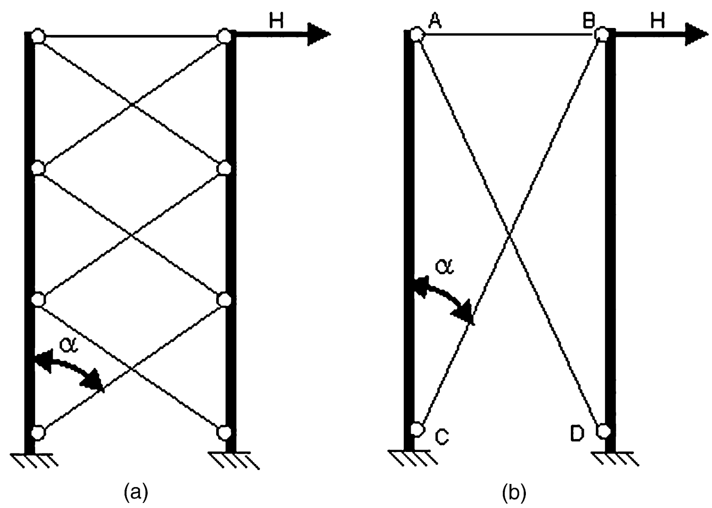


FIG. C2-5. Braced Systems Under Shear Load: (a) Stiff Bracing; (b) Insufficiently Stiff Bracing

Guyed structures and latticed H-frames may include masts built up with angles at the corners and lacing in the faces, as shown in Fig. C2-6. The overall cross section of the mast is either square, rectangular, or triangular. Latticed masts typically include a large number of members and are relatively slender, that is, they may be susceptible to second-order stresses. One alternative to modeling a mast as a three-dimensional truss system is to represent it by a model made up of one or several equivalent beams. The properties of an equivalent beam that deflects under shear and moment can be worked out from structural analysis principles. The beams are connected to form a three-dimensional model of the mast or an entire structure. That model may be analyzed with any three-dimensional finite element computer program. If large deflections are expected, a second-order (geometrically nonlinear) analysis should be used. Once the axial loads, shears, and moments are determined in each equivalent beam, they can be converted into axial loads in the members that make up the masts.

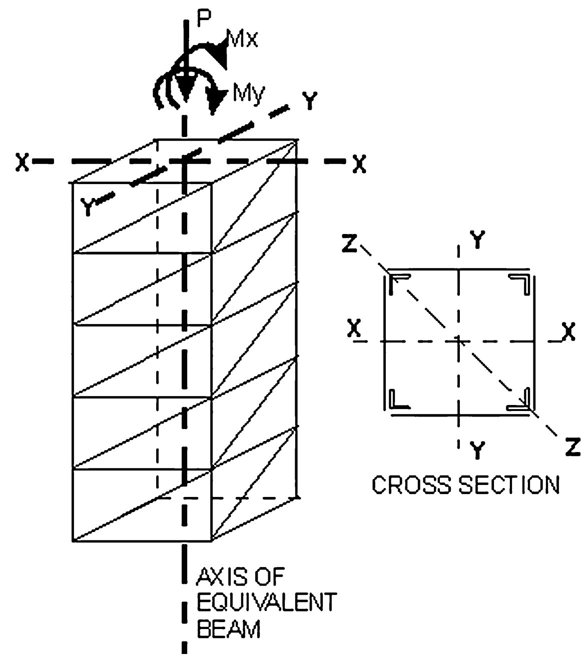


FIG. C2-6. Segment of Latticed Mast Idealized as Beam

CHAPTER C3
DESIGN OF MEMBERS

C3.1 INTRODUCTION

This standard is suitable for steels with yield points up to 65 ksi (448 MPa) and for width-to-thickness values of 25 for projecting elements, such as angle legs and channel flanges. The recommendations are intended for both hot-rolled and cold-formed members. Recommendations have also been included covering guyed transmission structures.

Test experience may indicate that these recommendations are conservative for specific shapes or connections. Higher values may be used where they are verified by tests, provided that the results are adjusted to the ASTM yield and tensile values of the material and for differences between the nominal design dimensions of the member and the actual cross-sectional dimensions of the test specimens. The properties of the material should be determined by tests on standard coupons taken in accordance with the requirements of AISC 360 (2010).

Designers should verify the availability of material section sizes to ensure economical design solutions because published section sizes may not be commercially available. Information is available from AISC and AISI.

C3.2 MATERIAL

A ratio of $F_u/F_y \geq 1.15$ is suggested for steel used for members.

C3.3 MINIMUM SIZES

If weathering steel is to be used, consideration should be given to increasing the suggested minimum thickness.

C3.4 SLENDERNESS RATIOS

Tension-only members used as cross-brace members and tension-hanger members are not designed to resist compression forces. Tension-only member performance is accomplished by maintaining high L/r values for these members. When tension-only members are subjected to compressive forces, they bow out-of-plane without permanent set and the compressive forces are redistributed to other members of the structural system. Upon removal of the applied compressive load, the tension-only member returns to its original position. Experience has shown that maintaining or limiting L/r values of 300 or greater accomplishes the desired performance.

In special cases based on structure geometry and loads, some tension-hanger members may not be subjected to compressive forces. For this case, lower L/r members could be used with caution and are allowed under the provisions of Section 3.18 (Test Verification); however, L/r values less than 300 for tension-only members are not appropriate in a general provision.

Damaging vibration of steel members in latticed towers usually occurs at wind speeds less than 20 mi/h (32 km/h) since a nearly constant velocity is required to sustain damaging vibration. Tests on a number of shapes with L/r values of 250 shows that the possibility of damaging vibration is minimal (Carpena and Diana 1971). Tension-hanger members are prone to vibration, but L/r values as large as 375 have been used successfully.

In areas of steady winds over extended periods, such as mountain passes or flat plains, allowable L/r values may need to be reduced. Where severe vibration is a concern, careful attention should be given to framing details. The practice of blocking the outstanding leg of angles to facilitate the connection should be avoided.

C3.5 PROPERTIES OF SECTIONS

Evaluation of torsional-flexural buckling involves some properties of the cross section that are not encountered in flexural buckling. Procedures for computing the torsional constant J, the warping constant C_w, the shear center, and other properties are given in Timoshenko and Gere (1961), Yu (1986), *Cold-Formed Steel Design Manual* (AISI 2008), and other sources. The AISC *Steel Construction Manual* (2011) gives tabular values for various standard shapes.

For cold-formed shapes with small inside-bend radii (twice the thickness), section properties can be determined on the basis of square corners. Equations based on round corners are given in the *Cold-Formed Steel Design Manual* (AISI 2008). Normally, the differences in properties based on square and round corners are not significant.

C3.6 DESIGN COMPRESSION

The Structural Stability Research Council (SSRC) formula, originally developed in 1966, for the ultimate strength of the centrally loaded column in the inelastic range and the Euler formula in the elastic range are used in this Standard. Test experience on tower members is limited in the range of L/r from 0 to 50, but indications are that the SSRC formula applies equally well in this range if concentric framing details are used.

C3.7 COMPRESSION MEMBERS: ANGLES

A single angle in axial compression can fail by torsional-flexural buckling, which is a combination of torsional buckling and flexural buckling about the u-axis; by z-axis flexural buckling, Fig. C3-1; or by local buckling of the legs. Local buckling and purely torsional buckling are identical if the angle has equal legs and is simply supported and free to warp at each end;

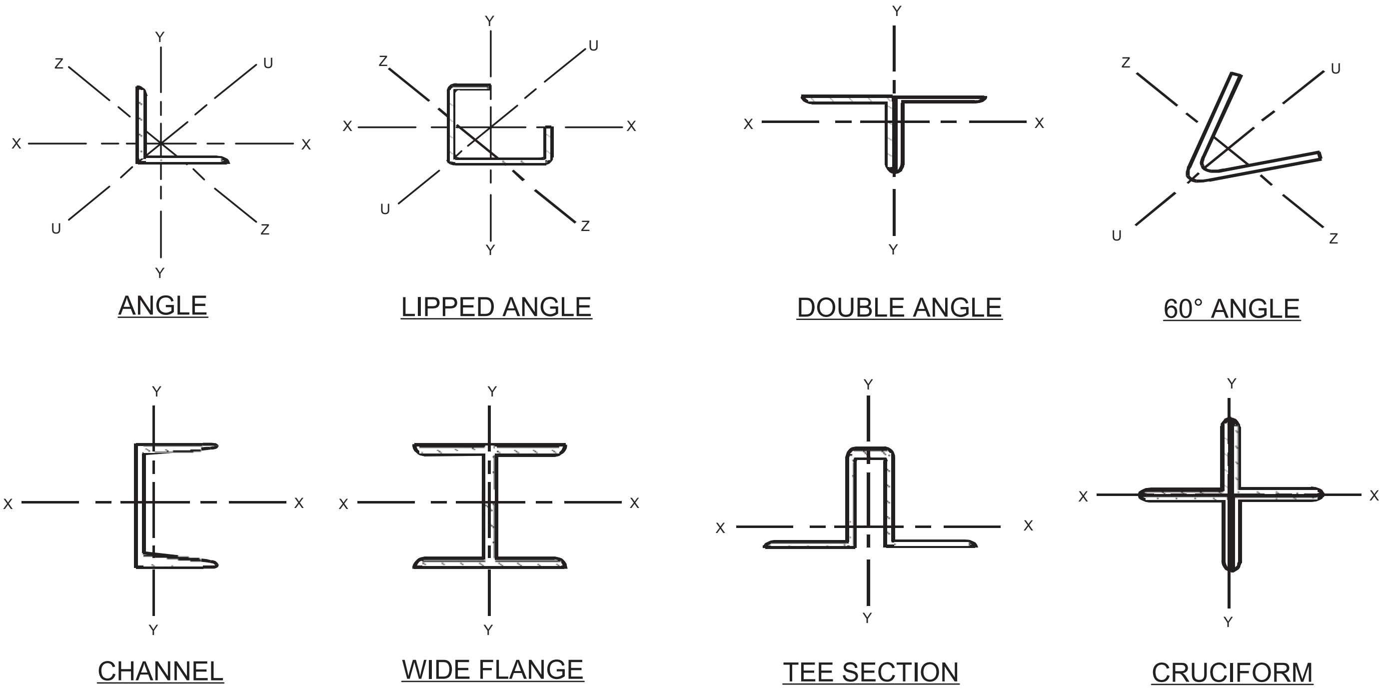


FIG. C3-1. Typical Cross Sections and Axes

furthermore, the critical stress for torsional-flexural buckling is only slightly smaller than the critical stress for purely torsional buckling, and for this reason such members have been customarily checked only for flexural and local buckling.

C3.7.3 Determination of F_a The ratio w/t of flat width to thicknesses that enables the leg to reach yield stress without buckling locally has been set at $80/\sqrt{F_y}$ for F_y in ksi ($210/\sqrt{F_y}$ for F_y in MPa). The reduced strength of legs with larger values of w/t is given by Eqs. (3.7-2) and (3.7-3). The effect of the reduced local-buckling strength on the flexural-buckling strength is accounted for by substituting the reduced value F_{cr} for F_y in Eqs. (3.6-1) and (3.6-3). Unequal-leg angles can be designed following this procedure by establishing the design stress based on the w/t value of the long leg. Member strengths computed by this procedure are in good agreement with test results on both hot-rolled and cold-formed single angle members (Gaylord and Wilhoite 1985).

C3.7.4 Effective Lengths The K factors for angle members depend upon the connection design for the member. The effective length of leg sections that have bolted connections in both legs is assumed to be the actual length ($K = 1$). In the design of members, the length (L) is usually determined from working point to working point on the design drawings. For other angles in compression, eccentricity of the connection is the predominant factor in the lower L/r range and is accounted for by specifying effective slenderness values KL/r. In the higher L/r range, rotational restraint of the members becomes the predominant factor and is also accounted for by specifying effective slenderness values KL/r. The break point is taken as $L/r = 120$. Member strengths computed by this procedure are in close agreement with numerous tests on both hot-rolled and cold-formed angles (Gaylord and Wilhoite 1985). Eccentricities at leg splices should be minimized, and the thicker sections should be properly butt-spliced.

Fig. C3-2 is a plot of normalized compression force versus L/r curves. This figure is provided solely for the purpose of understanding the effects of the different boundary conditions used in the compression capacity equations of this Standard. Equation (3.7-5) is the SSRC (Structural Stability Research Council) basic formula for $0 \leq L/r \leq 120$. Equation (3.7-8) is the Euler formula for $L/r \geq C_c$. Equations (3.7-6), (3.7-7), (3.7-9), and (3.7-10) are modifications of the basic curves. Equation (3.7-6) applies to angle members with concentric load at only one end. Equation (3.7-7) applies to members with framing eccentricities at both ends. Equation (3.7-9) applies to members partially restrained against rotation at one end. Equation (3.7-10) applies to members partially restrained against rotation at both ends. All these equations are assumed to have a common point at L/r of approximately 120. This L/r value represents the transition point in the compression capacity equations where eccentricity ($L/r \leq 120$) versus end restraint ($L/r \geq 120$) has the greater influence.

Note: Curve numbers (Fig. C3-2) have been included on the graph since these equations are often identified using these numbers and used in ASCE Manual of Practice No. 52 (1988).

Angle members connected by one leg should have the centroid of the bolt pattern located as close to the centroid of the angle as practical. Except for some of the smaller angles, normal framing eccentricity implies that the centroid of the bolt pattern is located between the heel of the angle and the centerline of the connected leg. When this is not the case, due consideration should be given to the additional stresses induced in the member.

To justify using the values of KL/r specified in Eqs. (3.7-9), (3.7-10), (3.7-12), and (3.7-13), the following evaluation is suggested:

(a) The restrained member should be connected to the restraining member with at least two bolts; and
(b) The restraining member should have a stiffness factor I/L in the stress plane (I = moment of inertia and L =

length) that equals or exceeds the sum of the stiffness factors in the stress plane of the restrained members that are connected to it. An example is shown in Fig. C3-3; and

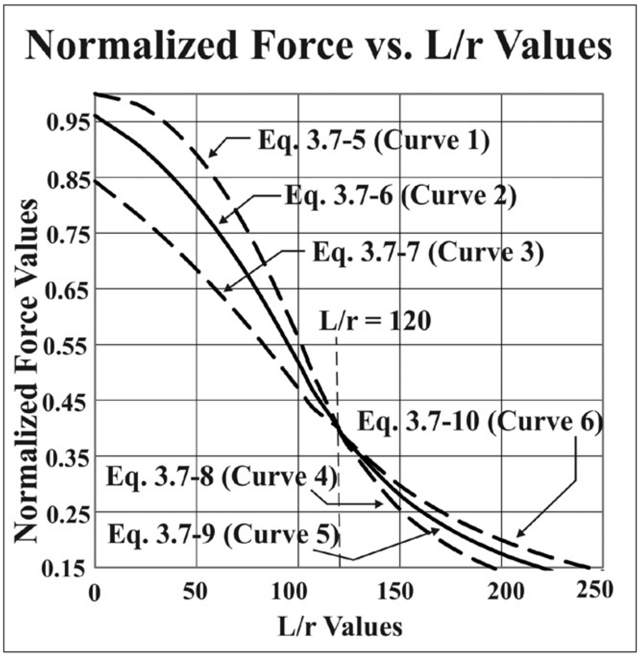


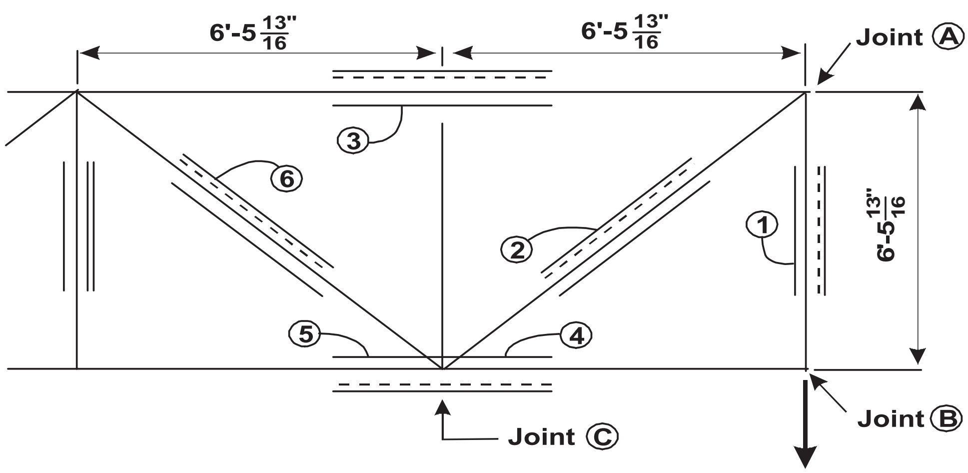
FIG. C3-2. Normalized Compression Curves

(c) Angle members connected solely to a gusset plate should not be considered to have end restraint against rotation. An angle member with an end connection to both a gusset plate and the restraining angle member should have adequate bolts in the restraining angle member to provide end restraint against rotation.

The critical loading condition for compression members being analyzed and the members providing joint restraint should be compared. If all members at a joint can buckle at the same time, then the joint cannot provide restraint for any of the members framing into it.

The L/r limitations specified in Section 3.7.4.3 normally ensure that the redundant member is adequate to provide support for the stressed member. Studies indicate that the magnitude of the redundant support required is dependent on the initial crookedness and the L/r value of the supported member.

C3.7.4.4 Unsupported Length with Varying Forces The effective length factor (K) for an unsupported length of a member with varying force magnitudes such as for the bracing systems shown in Fig. C3-4 can be determined using Eqs. (3.7-14a) and (3.7-14b) (Johnston 1988). The change in force at the subpanel point has the effect of providing partial restraint for both rotation and translation at this point. When out-of-plane support is provided and the load in the member is changing, each subpanel of the member should be designed as an individual member with due consideration of any eccentricities when selecting the value of KL/r.



$I = r^2A$ and $I/L = (rA)/(L/r)$

1 $L2 \times 2 \times 1/8$; one bolt connection—joint restraint not a consideration—tension member

2 and 6 $L2½ \times 2½ \times 3/16$; two bolt connection; $L = 110$ in.; $r_{xx} = 0.778$ in.; $A = 0.902$ in.^{2}

$$\frac{rA}{L/r} = \frac{0.778 \times 0.902}{110/0.778} = 0.005 \text{ (in truss plane)}$$

3 $L2½ \times 2½ \times 3/16$; $L = 155.6$ in.; $r_{xx} = 0.778$ in.; $A = 0.902$ in.^{2}

$$\frac{rA}{L/r} = \frac{0.778 \times 0.902}{155.6/0.778} = 0.004 \text{ (in truss plane)}$$

Joint restraint not a consideration—tension member.

4 and 5 $L4 \times 4 \times 1/4$; $L = 77.8$ in.; $r_{xx} = 1.25$ in.; $A = 1.94$ in.^{2}

$$\frac{rA}{L/r} = \frac{1.25 \times 1.94}{77.8/1.25} = 0.039 \text{ (in truss plane)}$$

Joint (A): 3 < 2, 0.004 < 0.005, no restraint for 2.
Joint (B): Single bolt connection—no restraint.
Joint (C): 4 + 5 > 2 + 6, 2 × 0.039 > 2 × 0.005—Partial restraint for 2 and 6 at this joint.
Member 2—Partial restraint at one end; $r_{zz} = 0.495$ in.;
Eq. (3.7-9) $L/r_{zz} = 110/0.495 = 222.2$;
$KL/r_{zz} = 28.6 + 0.762 \times 222.2 = 198$—Member meets requirements of compression member.

FIG. C3-3. Member Restraint Determination (All dimensions in inches: 1 in. = 25.4 mm)

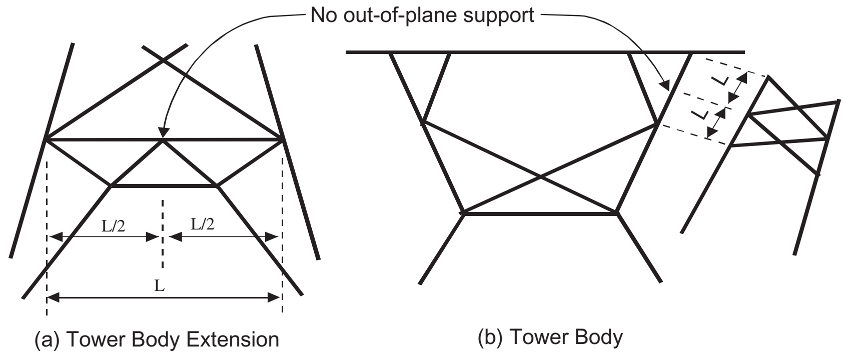


FIG. C3-4. Typical Subpanel Configurations

C3.8 COMPRESSION MEMBERS: SYMMETRICAL LIPPED ANGLES

Lips increase the local buckling strength of the legs of an angle, and in some applications lipped angles show an advantage over plain angles. Since the local buckling strength of the angle is not equivalent to torsional buckling of the angle, torsional-flexural buckling should be considered. The design compressive stress for torsional-flexural buckling is determined by using an equivalent radius of gyration r_{tf} [Eq. (3.8-1)] in the design column stress formulas [Eqs. (3.6-1) and (3.6-2)].

The effective-length coefficient in Eqs. (3.6-1), (3.6-2), and (3.8-2) is $K = 1$ if the member is free to warp and to rotate about the u-axis at each end.

If warping and u-axis rotation are prevented at both ends, $K = 0.5$; if they are prevented at only one end, $K = 0.7$. Mixed end conditions can be treated by replacing r_t and r_u in Eq. (3.8-1) with r_t/K_t and r_u/K_u, where K_t and K_u are the effective-length coefficients for torsional and u-axis buckling, respectively. Eq. (3.8-1) in this form gives the value of K/r_{tf} by which L is multiplied for use in Eqs. (3.6-1) and (3.6-2). However, $K_t = 1$ should be used in Eq. (3.8-2) when it is used to compute the adjusted values KL/r specified in Section 3.7.4. Gaylord and Wilhoite (1985) and Zavelani and Faggiano (1985) provide additional test verifications.

If there are no intermediate supports, the design stress is given by Eqs. (3.6-1) and (3.6-2), using for KL/r the larger of KL/r_z and KL/r_{tf}. If there are intermediate supports, the length L used to determine the slenderness ratio, depends on the nature of the support, that is, whether it restrains only flexural buckling, only torsional-flexural buckling, or both.

C3.9 COMPRESSION MEMBERS NOT COVERED IN SECTIONS 3.7 AND 3.8

C3.9.2 Maximum *w/t* Ratio Most of the shapes other than angles that are likely to be used in transmission towers have element slenderness ratios, w/t, small enough to develop a uniform distribution of the stress F_a given by Eqs. (3.6-1) and (3.6-2) over the full cross-sectional area. Where this is not the case, the postbuckling strength of elements that buckle prematurely is taken into account by using an effective width of the element in determining the area of the member cross section. The effective width of an element is the width that gives the same resultant force at a uniformly distributed stress F_a as the nonuniform stress that develops in the entire element in the postbuckled state.

C3.9.3 Effective Widths of Elements in Compression Effective widths in this section are derived from formulas in *North American Specification for the Design of Cold-Formed Steel Structural Members* (AISI 2007). Only the effective widths of Section 3.9.3.1 are needed for the uniform stress distribution in axially loaded compression members.

Stress gradients (Section 3.9.3.2) occur in members in bending, and effective widths for this case are needed only for beams and eccentrically loaded compression members.

The effective widths for axially loaded members are determined at the design stress F_a based on the radius of gyration of the gross cross section, whereas the allowable force P is obtained by multiplying F_a by the gross area if all elements are fully effective and by the reduced area if the effective widths of the elements are smaller than the actual widths.

The types of buckling that should be checked for axially loaded members with symmetric cross sections are covered in Sections 3.9.4–3.9.7. For members that may be subject to torsional or torsional-flexural buckling, an equivalent radius of gyration, r_t, for doubly symmetric and point-symmetric sections and r_{tf} for singly symmetric sections are specified to be used in determining the design stress F_a by Eqs. (3.6-1) and (3.6-2). Note that r_{tf} and r_t [Eqs. (3.8-1) and (3.8-2)] are referred to as the principal axes u and z of angles (Fig. C3-1). In adjusting the formulas for the x and y principal axes of other sections, u is the axis of symmetry. Therefore, when either the x-axis or the y-axis is the axis of symmetry, it should be substituted for u in Eq. (3.8-1), and also whenever it appears in the list of symbols following Eq. (3.8-2).

C3.9.8 Nonsymmetric Cross Sections An analysis based on the elastic buckling stress of a nonsymmetric member, which requires the solution of a cubic equation, is suggested in Zavelani and Faggiano (1985). In general, this analysis gives an upper bound to the allowable value. A lower bound can be obtained by proportioning the member so that the maximum combined stress caused by the axial load and moment equals the yield stress (Madugula and Ray 1984).

C3.10 TENSION MEMBERS

C3.10.5 Guys The elastic limit of wire rope is approximately 65% of its strength; therefore, the specified 0.65 times the minimum breaking strength is consistent with this standard's ultimate stress F_y for tension members. The factor of safety of a guy with respect to its elastic limit is equal to the load factor, as with other structural components.

The determination of the tension of guys should be based on the movement of the guy anchor under load, the length and size of the guy, the allowable deflection of the structure, and the modulus of the guy. On tangent structures, pretensioning of guys to 10% of their rated breaking strength is normally sufficient to avoid a slack guy.

C3.12 AXIAL COMPRESSION AND BENDING

Equations (3.12-1) and (3.12-2) are the same as the corresponding formulas in the AISC Allowable Stress Design specification and the AISI specification, except that in the AISC specification they are given in terms of axial stress f_a and bending stress f_b instead of force P and moment M. Values of C_m are given here only for the case where there is no lateral displacement of one end relative to the other, since the lateral-displacement (sideways) case is not likely to be found in latticed transmission towers.

Both the AISC and AISI specifications give an alternative simplified formula, which may be used if f_a/F_a is less than 0.15. Because this case is likely to be rare in transmission tower work, the corresponding formula is not given in this standard.

C3.13 AXIAL TENSION AND BENDING

The term $1/(1 - P/P_e)$ in Eq. (3.12-1) accounts for the increase in the moments M_x and M_y due to the eccentricity of the compression force P caused by the bending of the member. If the axial force is tension, however, its effect decreases the moments. Therefore, the inclusion of terms in Eq. (3.13-1) for decreasing the moments would be logical. However, the decrease is not usually accounted for and the resulting simpler formula is used in this standard.

M_{ax} and M_{ay} are to be determined according to Section 3.14. Therefore, the effects of lateral buckling for members not supported laterally are taken into account even though the lateral-buckling moment is based on a compressive stress. In other words, Eq. (3.13-1) is not based on the addition of axial tension and tensile stresses due to bending. This logic can be seen by considering the case in which P and M_x are both zero. This case gives $M_y = M_{ay}$, and if the member is not supported against lateral buckling, M_{ay} should be the lateral-buckling moment.

Eq. (3.13-1) is the same as the corresponding equation in AISC 360 (2010).

C3.14 BEAMS

Formulas in this section for determining allowable moments differ in form from those in the AISC and AISI specifications in that the allowable compressive stress for laterally unsupported members is computed from the allowable stress formulas for axial compression through the use of an equivalent radius of gyration.

C3.14.4 I, Channel, and Cruciform Sections Eq. (3.14-1), which gives the equivalent radius of gyration for doubly symmetric I sections, symmetric channels, and singly or doubly symmetric cruciform sections, takes both the St. Venant torsional stiffness J and the warping stiffness, C_w, into account. Values of $K = \sqrt{K_y K_t}$ and C_b for a number of cases for which the member end conditions are the same for warping and y-axis rotation (e.g., warping and y-axis rotation both permitted or both prevented) are given by Clark and Hill (1960) and Gaylord et al. (1992). Two formulas are used in AISC 360 (2010). One is obtained by taking $C_w = 0$ and the other by taking $J = 0$ and expressing the results in terms of other, more familiar properties of the cross section. The larger of the two design stresses so obtained is used because both underestimate the buckling strength because of the omissions just mentioned. The two formulas can be used only for doubly symmetric I sections and singly symmetric I sections with the compression flange larger than the tension flange. Only one of the two applies to channels. On the other hand, formulas in the *North American Specification for the Design of Cold-Formed Steel Structural Members* (AISI 2007) are derived by assuming that $J = 0$ because it is usually relatively small for thin-walled shapes of cold-formed members. However, Eq. (3.14-1) is not difficult to use, and it has the advantage of giving more accurate values of the buckling stress.

C3.14.6 Singly Symmetric I and T Sections The approximate procedures for T sections and singly symmetric I sections give very good results.

C3.14.7 Other Singly Symmetric Open Sections The formulas in this section are expressed in different terms from those in the AISI (2008) specification. However, they give identical results. Gaylord et al. (1992) give a typical example.

C3.14.8 Equal Leg Angles The formulas in this section give critical moments for pure bending (constant moment) of equal leg angles and therefore are conservative for cases where there is a moment gradient, as for a uniformly loaded beam. However, they do not account for twisting of the angle, so they are unconservative if the load does not act through the shear center. Tables of allowable uniformly distributed load perpendicular to a leg, based on formulas that account for twist due to load eccentricity of $\pm b/2$, which have been confirmed by an extensive series of tests, are available (Leigh et al. 1984 and Madugula and Kennedy 1985). The tables cover angles with unequal legs, as well as those with equal legs. The loads are based on an allowable stress of 0.6 F_y in Madugula and Kennedy (1985), and 0.66 F_y in Leigh et al. (1984), so that the tabulated values should be divided by 0.6 and 0.66, respectively, to obtain values in conformity with tower design practice.

Predictions of the formulas in this section for load perpendicular to a leg are in good agreement with the values in Leigh et al. (1984) for beam spans up to $L/r_z = 250$. The tabular values range from about 92% for the relatively thin angles ($b/t = 16$) to about 115% (b/t about 8) of the formula values using the centerline section moduli of Eq. (3.14-11), and from about 94 to 130% using overall dimension section moduli. The overprediction of the thin angles is partially compensated for by the fact that the formulas give the critical uniform moment. Nevertheless, if the formulas are used for angles with b/t larger than 16, it is suggested that the resulting allowable moments be reduced by 10% if the member is not to be tested.

Deflections may be computed as the resultant of the u- and z-axis deflection components determined by resolving the moment M into the u- and z-axis. There are no established limits of deflection for beams in transmission tower applications.

C3.15 DESIGN SHEAR

C3.15.1 Beam Webs The upper limit of h/t in the AISC specification (2010) is given by a formula involving F_y. The limiting value 200 of this section, which is the same as the AISI (2008) specification limit, equals the AISI limit for $F_y = 60$ ksi (413 MPa) and is, therefore, adequate for steels with $F_y \leq 60$ ksi (413 MPa). It is unlikely that webs thinner than those allowed by the 200 limit are needed in transmission towers.

The design shear stresses given here are the same as those in the AISC and AISI specifications multiplied by the AISC and AISI factors of safety.

C3.16 REDUNDANT MEMBERS

Redundant members provide intermediate support for stressed members. Common design practice is to design redundant members for 1.5% to 2.5% of the load in the supported member.

C3.17 WELDED ANGLES

Welded angles are seldom used but are sometimes necessary for single-angle members (e.g., greater than 8 in. × 8 in. or odd-size angles). Welded angles should be fabricated using steel that is suitable for welding and galvanizing.

CHAPTER C4
DESIGN OF CONNECTIONS

C4.1 INTRODUCTION

The purchaser's procurement specifications should specify if the end and edge distances are minimum values that cannot be underrun. Tolerances for sheared and cut ends are normally established by the supplier. Edge distances are controlled by the gauge lines selected, and the detailer should provide for normal rolling tolerances to avoid possible underrun of the edge distances. The rolling tolerances contained in ASTM Standard A6 (2012) should be used as a guide.

Bolts, such as those in ASTM A394 (2008), are installed to the snug-tight condition or to some specified minimum torque. Even if the bolt is supplied with a lubricant, it is difficult to fully torque a hot-dipped galvanized bolt because of the buildup of the zinc coating on the threads as the nut is tightened. Consequently, locking devices are used by many utilities to minimize possible loosening of the nut caused by vibration or flexure of the structure joints.

C4.3 FASTENERS

C4.3.2 Bolt Shear Capacity Bolts, such as A394 bolts (ASTM 2008) are typically installed to the snug-tight condition or to some specified minimum torque. Thus, the load transfer across a bolt is governed by direct shear rather than friction. ASTM A394 (ASTM 2008) provides the specified minimum shear values when threads are included in or excluded from the shear plane. The design shear of $0.62F_u$ for bolts that do not have an ASTM-specified shear stress is conservative (Kulak et al. 1988).

C4.3.3 Bolt Tension Capacity The specified tensile stresses approximate those at which the rate of elongation of the bolt begins to increase significantly. The ASTM proof load stress is approximately equal to the yield stress, and $0.6F_u$ is a conservative estimate for bolts for which the proof load is not specified.

If design stresses exceed the yield stress, permanent stretch can occur in the bolt. This stretch could loosen the nut and cause a loss of tightness in the joint.

C4.3.4 Bolts Subject to Combined Shear and Tension Tests on rivets and bolts indicate that the interaction between shear and tension in the fastener may be represented by formulas that plot as ellipses (Higgins and Munse 1952; Chesson et al. 1965; and Kulak et al. 1988). Therefore, an elliptical expression, with major and minor axes based upon the design shear and tension values given in Sections 4.3.2 and 4.3.3, has been specified.

C4.4 DESIGN BEARING STRESS

The design bearing stress is the same as the allowable value in the AISC 360 (2010) specification. This value may seem unduly conservative since in the AISC specification the allowable value is applied to the service (unfactored) load whereas in the Standard it is applied to the factored load. However, it conforms to experience in the tower industry. Designs produced with bearing values less than or equal to $1.5F_u$ and conforming with the other provisions of this document have demonstrated satisfactory control over bolt-hole ovalization during full-scale tower tests. Furthermore, the AISC value is reduced to $1.2F_u$ in the 2010 edition of the specification if deformation around the hole is a consideration.

When applying these provisions, the designer should recognize that the required end and edge distances depend upon the bearing stress in the connection. It may be useful to reduce the bearing stress below the maximum allowed design value because this value may permit a reduction in the end and edge distances required.

The design bearing value on the bolts should be checked if the tensile strength, F_u, of the member material exceeds the F_u value of the bolt. This would occur if an A394 (ASTM 2008), Type 0 bolt ($F_u = 74\,\text{ksi} = 510\,\text{MPa}$) is used to connect an A572-Grade 65 material ($F_u = 80\,\text{ksi} = 551\,\text{MPa}$). The full diameter of the bolt should be used for this calculation, with design bearing stress equal to $1.5F_u$ of the bolt (Wilhoite 1986).

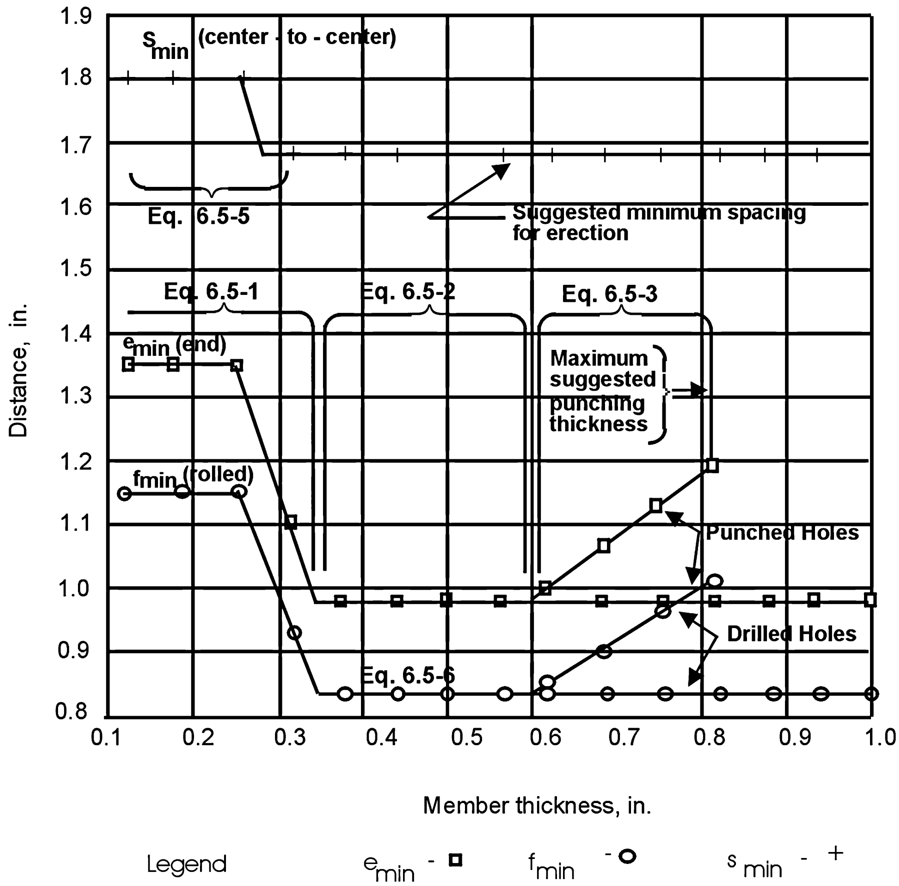
C4.5 MINIMUM DISTANCES

C4.5.1 End Distance (See Fig. C4-2) The provisions of this section are applicable to sheared and mechanically guided flame-cut ends.

Eq. (4.5-1) provides the end distance required for strength. The required end distance is a function of the load being transferred in the bolt, the tensile strength of the connected part, and the thickness of the connected part. Test data confirm that relating the ratio of end distance to bolt diameter to the ratio of bearing stress to tensile strength gives a lower bound to the published test data for single fastener connections with standard holes (Kulak et al. 1988). The end distance required by the above expression has been multiplied by 1.2 to account for uncertainties in the end distance strength of the members (Kulak et al. 1988). For adequately spaced multiple bolt connections, this expression is conservative.

Eq. (4.5-2) is a lower bound on end distance that has been successfully used in tower practice in stressed members. A minimum end distance of $1.2d$ has been specified for redundant members since they carry only secondary stresses, which are much less than stresses in the members they brace.

Latitude is provided to use the minimum end distances and determine the design bearing stress for this condition. Eqs. (4.5-1) and (4.5-2) allow one to determine what combination of bearing value and end distance satisfies the engineering and



End, rolled edge and center to center distances for a A394, Type O, 3/4 in. diameter bolt used to connect A36 stressed steel members. Bolt in single shear (through threads). Bearing stress = 1.5 Fu

FIG. C4-1. A394 Type 0 Bolt Connecting A36 Steel (1 in. = 25.4 mm)

detailing requirements. Eq. (4.5-3) places an end distance restriction on thick members such that punching of the holes does not create a possible breakout condition. If the holes are drilled in members where the end distance would be governed by Eq. (4.5-3), this requirement is not necessary. Satisfactory punching of the holes in thick material is dependent on the ductility of the steel, the adequacy of the equipment (capability of the punching equipment and proper maintenance of punches and dies), the allowed tolerances between the punch and die, and the temperature of the steel. The following guidelines have been used satisfactorily:

For 36 ksi (248 MPa) yield steel, the thickness of the material should not exceed the hole diameter;

For 50 ksi (345 MPa) yield steel, the thickness of the material should not exceed the hole diameter minus 1/16 in. (1.6 mm); and

For 65 ksi (448 MPa) yield steel, the thickness of the material should not exceed the hole diameter minus 1/8 in. (3.2 mm).

Fig. C4-1 shows the end distances for an A394, Type 0, 3/4 in. (19 mm) diameter bolt used to connect A36 steel stressed members. The maximum bearing stress of $1.5F_u$ has been used unless shear governs the maximum force in the bolt. Table C4-1 contains the tabulated values for Fig. C4-1 and shows that Eq. (4.5-1) governs up through a thickness of 5/16 in. (6.9 mm); Eq. (4.5-2) governs for thicknesses of 3/8 in. (9.5 mm) through 9/16 in. (14.3 mm); and Eq. (4.5-3) governs thicknesses of 5/8 in. (15.9 mm) and above, unless the holes are drilled. For drilled holes, Eq. (4.5-2) would continue to govern over this range of thicknesses. The range of thicknesses over which each equation governs changes if the bearing stress is reduced below the maximum design bearing stress of $1.5F_u$. Fig. C4-2 illustrates the proper application of the required end distance when a member is clipped and the sheared surface is not perpendicular to the primary axes of the member.

C4.5.2 Center-to-Center Bolt Hole Spacing Two factors should be considered when determining the minimum center-to-center bolt spacing. Eq. (4.5-5) is the strength expression for end distance [Eq. (4.5-1)] plus 0.6 times the diameter of the adjacent bolt. The term $0.6d$ has been specified instead of the more commonly recognized $0.5d$ to provide greater control over the reduction in center-to-center material because of hole breakout during punching.

For low bearing values, the spacing requirements predicted by Eq. (4.5-5) may be less than the spacing required for installation. Spacing requirements for convenient installation should be determined by adding 3/8 in. (9.5 mm) to the width across the points of the nut being used. Fig. C4-1 and Table C4-1 show that the installation requirement governs for thick members.

Table C4-1. A394 Type 0 Bolt Connecting A36 Steel (1 in. = 25.4 mm; 1 ksi = 6.89 MPa; 1 kip = 4,450 N)

Values for end, rolled edge, and center-to-center distances for an A394, Type 0, 3/4-in.-diameter bolt used in single shear through the threads to connect A36 steel stressed members. Bearing stress = 1.5F_u

Member Thickness t (in.) (1)	Bearing Stress 1.5F_u (ksi) (2)	Force Bolt P (kip) (3)	Eq. (4.5-1) e (in.) (4)	Eq. (4.5-2) e (in.) (5)	Eq. (4.5-3) e (in.) (6)	e_{min} (in.) (7)	Eq. (4.5-5) S_{min} (in.) (8)	Suggested Erection Spacing (in.) (9)	S_{min} (in.) (10)	Eq. (4.5-6) f_{min} (in.) (11)
1/8	87.0	8.16	1.35	0.98	0.50	1.35	1.80	1.67	1.80	1.15
3/16	87.0	12.23	1.35	0.98	0.56	1.35	1.80	1.67	1.80	1.15
1/4	87.0	16.31	1.35	0.98	0.63	1.35	1.80	1.67	1.80	1.15
5/16	87.0	16.65[a]	1.10	0.98	0.69	1.10	Less	1.67	1.67	0.94
3/8	87.0	16.65[a]	Less	0.98	0.75	0.98	Than	1.67	1.67	0.83
7/16	87.0	16.65[a]	Than	0.98	0.81	0.98	Col. 9	1.67	1.67	0.83
1/2	87.0	16.65[a]	Col. 5	0.98	0.88	0.98		1.67	1.67	0.83
9/16	87.0	16.65[a]		0.98	0.94	0.98		1.67	1.67	0.83
5/8	87.0	16.65[a]		0.98	1.00	1.00		1.67	1.67	0.85
11/16	87.0	16.65[a]		0.98	1.06	1.06		1.67	1.67	0.90
3/4	87.0	16.65[a]		0.98	1.13	1.13		1.67	1.67	0.96
13/16	87.0	16.65[a]		0.98	1.19	1.19		1.67	1.67	1.01
7/8	87.0	16.65[a]		0.98	0.98[b]	0.98[b]		1.67	1.67	0.83[b]
15/16	87.0	16.65[a]		0.98	0.98[b]	0.98[b]		1.67	1.67	0.83[b]
1	87.0	16.65[a]		0.98	0.98[b]	0.98[b]		1.67	1.67	0.83[b]

Eq. (4.5-1) = $1.2P/(F_u t)$
Eq. (4.5-2) = $1.3d$
Eq. (4.5-3) = $t + d/2$
Eq. (4.5-5) = $1.2P/(F_u t) + 0.6d$
Erection = nut c dimension + 0.375
Eq. (4.5-6) = $0.85e_{min}$
Bolt diameter = 0.75 in.
Bolt shear strength = 16.65 kip
Point-to-point nut dimension = 1.3 in.
Member tensile strength (F_u = 58.0 ksi)

[a] P limited by single shear strength (through threads) of bolt.
[b] t = 13/16 in.; suggested maximum punching thickness (13/16-in. diameter hole in A36 steel). Distance shown is for drilled holes.

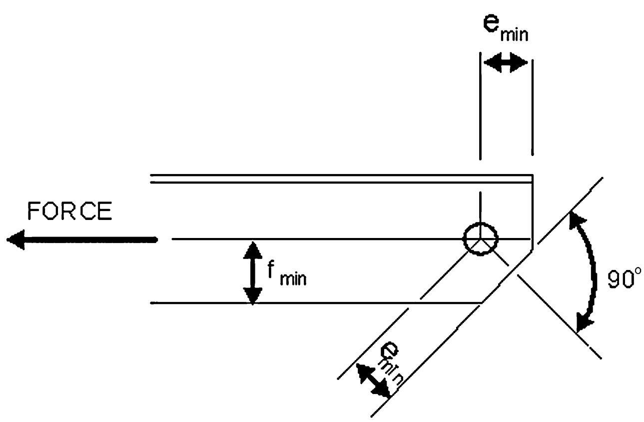


FIG. C4-2. Application of e_{min} to Clipped Member

C4.5.3 Edge Distance (See Fig. C4-2) Several studies have considered the effects of end and edge distance on the strength of connections for stressed members (Kennedy and Sinclair 1969; Gilchrist and Chong 1979; and Bodegom et al. 1984). Tests by Bodegom et al. (1984) establish a ratio of the rolled edge distance to the sheared edge distance to prevent a tension tear-out of the rolled edge. Eq. (4.5-6) combines the results given in Bodegom et al. (1984) with the minimum end distances in Eq. (4.5-2). The same ratio is retained for determining the relationship between all end distances and the required rolled edge distance. When the edge distance is to a sheared or mechanically guided flame-cut edge, 1/16 in. (1.6 mm) is added to the rolled edge distance. Fig. C4-1 shows the relationship between the rolled edge and end distances.

C4.6 ATTACHMENT HOLES

Oversized holes are commonly used as load attachment points for insulator strings, overhead ground wires, and guys. These holes are not used where connections are designed for load reversal. The possible failure modes considered are bearing, tension, and shear. These recommendations do not exclude the use of other attachment holes or slots designed by rational analysis.

Eq. (4.6-1) assumes that the member will fail in shear with shear planes developing at each side of the bolt through the edge of the member; Fig. C4-3 illustrates the various terms used. The equation was developed by replacing e in Eq. (4.5-1) with $(L - 0.5d_h)$, solving for P, and multiplying the result by 0.9. Given that L is defined as the distance from the center to any member edge, the member always fails in shear before it fails in tension. The dimension L shown for the edge distance, perpendicular to the line of action of P, may be reduced if analysis shows that the sum of the tensile stress P/A and the tensile bending stress does not exceed $0.67F_y$, where F_y is the member yield stress.

Eq. (4.6-1) has been limited to hole diameters less than or equal to two bolt diameters. This value represents the range of experience over which the equation has been used in practice. No adjustment to Eq. (4.6-1) is required for slight chamfering of the hole; however, where greater amounts of chamfering are used, the reduction in the effective thickness for bearing on the bolt and plate should be considered.

For attachment plates subject to bending, additional analysis is required to determine the plate thickness. Eq. (4.6-2) limits the bearing stress to 0.9 times the design value for standard holes to provide an additional factor for possible wear.

For everyday loading, as specified by the purchaser, P should not exceed $0.5dtF_u$ to avoid indentation of the material under

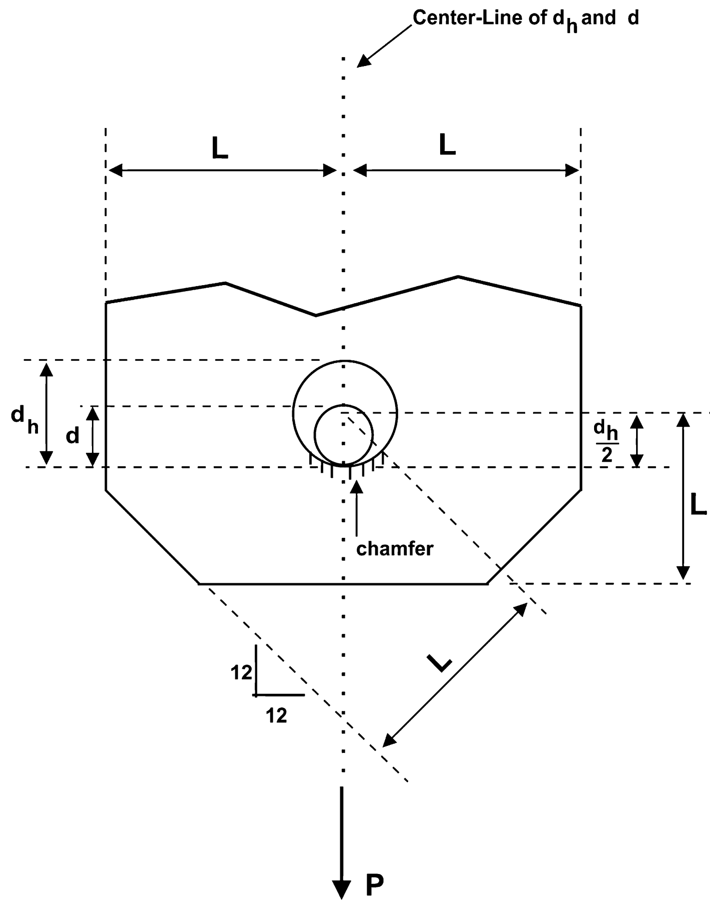


FIG. C4-3. Application of Oversized Holes

sustained loading and excessive wear. Everyday loading may be defined as that resulting from the bare weight of the conductor at 60°F (16°C) final sag, unless the location is subjected to steady prevailing wind. If the location is subjected to steady prevailing wind, the everyday loading may be considered to be the resultant load caused by the bare weight of the conductor and the prevailing wind at 60°F (16°C) final sag.

C4.7 POST ANGLE MEMBER SPLICES

There are two typical types of post angle member splices: butt and lap. Butt splices are commonly used, but lap splices have also been extensively used with good structural performance.

Well-designed butt splices transfer the axial loads from one section of the post angle directly to the next section without creating appreciable bending moment. The design of a butt splice may be performed by the detailer and approved by the engineer of record (EOR), or each butt splice should be custom-designed by the EOR and shown on the design drawing.

A typical butt splice consists of an inside angle and two outside plates with all connection bolts working in double shear; however, it may also be designed using a combination of inside and outside plates.

When using inside angles and outside plates, the designer should consider the following:

- The gross and net cross-sectional areas of the splice material should be equal to or greater than the corresponding gross and net cross-sectional areas of the post angle,
- The area of the inside angle should be approximately equal to the area of the outside plates,
- The centroid of the combined splice material should coincide with the centroid of the post angle as closely as possible,
- The bolt patterns should minimize eccentricities in the transfer of stress. In most cases, the centroids of the bolt patterns should be at the middle of each leg of the post angles,
- The post angle bolt end distance should be kept to a minimum.

A similar approach may be used when inside and outside plates are used.

A lap splice creates a bending moment because of the eccentricity of the centroids of the members being connected; however, they are frequently found in towers that have been validated by full-scale testing. Lap splice connections of thinner standard angle sizes, b/t minimum range of 12 to 14, that are commonly used for post angles of lattice towers can be used with the appropriate axial load capacity equation provided in Section 3.7.4.2 of this Standard. Lap splice connections of thicker post angles create larger moments than thinner post angles and should be further downgraded in load capacity using the provisions of Section 3.12.

CHAPTER C5
DETAILING AND FABRICATION

C5.2 FABRICATION

C5.2.5 Welding

C5.2.5.2 Welded Angles Normally, welded angles should be fabricated using complete penetration groove welds that conform to all the requirements of AWS D1.1 (2010). The plate edges that form the toes of the angle should be smooth and straight and should have all sharp edges removed and/or deburred. As a minimum, it is recommended that the permissible variation in shape conform to ASTM A6 (2014) for hot-rolled angles. All welded angles should be mechanically straightened before galvanizing. The engineer of record should specify if stress relieving is required. The purchaser and fabricator should agree on the member straightness tolerances to be used as the basis for material acceptance. If partial weld penetration is used, the requirements of Section 3.17 would apply.

CHAPTER C6
TESTING

C6.1 INTRODUCTION

Although this Standard provides specific recommendations for determining the design strengths of individual members and connections, analysis techniques used to predict forces in individual members and connections are dependent upon assumptions relative to the distribution of forces and reactions between portions of the overall structure, so new or unique structure configurations are often tested to verify that the stress distribution in the members is in accordance with the analysis assumptions and that the members selected, and the connections as detailed, are adequate to carry the design loads. This type of test is referred to as a proof test and is performed on a prototype of the full structure or a component of the structure, usually before that structure or a structure of similar design is fabricated in quantity.

A traditional proof test is set up to conform to the design conditions, that is, only static loads are applied, the prototype has ideal foundations, and the restraints at the load points are the same as in the design model. This kind of test verifies the adequacy of the members and their connections to withstand the static design loads specified for that structure as an individual entity under controlled conditions. Proof tests can provide insight into actual stress distribution of unique configurations, fit-up verification, action of the structure in deflected positions, adequacy of connections, and other benefits. The test cannot confirm how the structure will react in the transmission line where the loads may be dynamic, where the foundations may be less than ideal, and where there is some restraint from intact wires at the load points.

The testing procedures provided in this document are based on performing a proof test using a test station that has facilities to anchor a single prototype to a suitable base; load and monitor pulling lines in the vertical, transverse, and longitudinal directions; and measure deflections.

C6.2 FOUNDATIONS

C6.2.1 General The type, rigidity, strengths, and moment reactions of the actual attachments of a prototype to a test bed have a major effect on the ability of the prototype members to resist the applied loads; consequently, the reactions of the test foundations should be similar to the expected reactions of the in-service foundations.

C6.2.2 Rigid Structures Tests of a fairly rigid, four-legged, latticed structure designed for stub angles set in concrete, anchor bolts and base plates on concrete, or earth grillage foundations, are usually performed on special stub angles bolted or welded to the test station's rigid base. Accurate positioning of the footings is necessary to prevent abnormal stresses in the structure members.

C6.2.3 Direct Embedded Structures It is recommended that structures designed for direct embedment be tested in a two-part program.

C6.2.3.1 Embedded Portion Soil properties at a permanent test station probably do not match the properties of the soil on the transmission line. Tests that are dependent on soil resistance should be done at the line site, since only load cases that control the anchorage design need to be performed.

C6.2.3.2 Aboveground Portion The aboveground portion of the prototype should be modified to be bolted or welded to the test station's foundation. All controlling load cases should be applied to this prototype.

C6.2.4 Components For component tests, especially single members, the amount of rotational rigidity of the supports is critical. All the other parameters of the attachments of the prototype to the test bed should also be as close to the in-service conditions as possible.

C6.3 MATERIAL

Proper interpretation of the data obtained from testing is critical in establishing the true capacity of individual members. There is concern about using members in a prototype that have yield points considerably higher than the minimum guaranteed yield value that is used as the basis for design. The actual yield points of tension members and of compression members with KL/r values less than 120 are critical in determining the member capacity. Consequently, the guidelines shown in Fig. C6-1 are suggested as a basis for determining the maximum yield point values for these members of the prototype. All other members of the prototype should conform to the standard material specifications, but their actual yield points are not as critical to their load-carrying capabilities.

Changes in steel grades of the test tower should be approved by both the engineer of record and the purchaser. If the original specified steel, for tension members or compression members with KL/r less than 120, is not used in the test tower, the design and detail drawings should be changed to reflect the grade of steel used during the test.

C6.4 FABRICATION

Normally, the prototype is not galvanized for the test unless the purchaser's order states otherwise.

C6.5 STRAIN MEASUREMENTS

Stress determination methods, primarily strain gauging, may be used to monitor the loads in individual members during testing. Comparison of the measured unit stress to the predicted unit

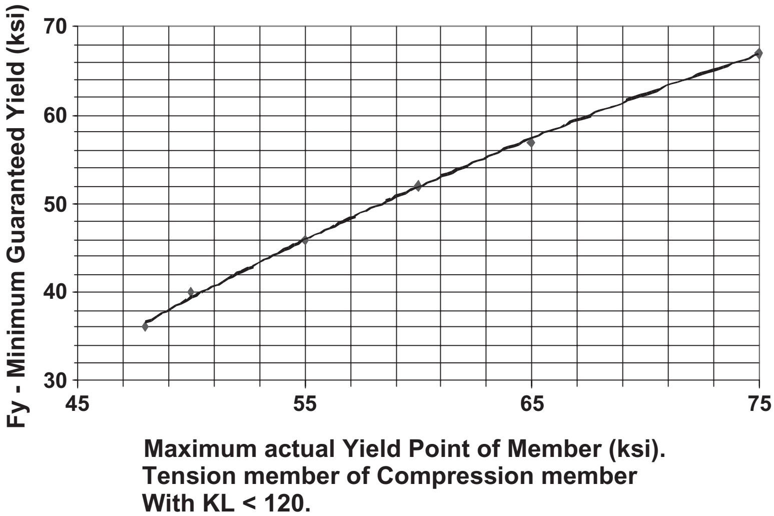


FIG. C6-1. Maximum Overstrengths for Members of Prototype (1 ksi = 6.89 MPa)

stress is useful in validating the proof test and refining analysis methods. Care should be exercised when instrumenting with strain gauges, both as to location and number, to ensure valid correlation with design stress levels.

C6.6 ASSEMBLY AND ERECTION

The erected prototype should conform with the special requirements of the purchaser's instructions; many purchasers specify both minimum torque for bolt tightening and that the vertical axis should not be out of plumb by more than 1 in. (25 mm) for every 40 ft (12 m) of height.

C6.8 LOAD APPLICATION

V-type insulator strings should be loaded at the point where the insulator strings intersect. If the insulators for the towers in the line are V-type strings that do not support compression, it is recommended that articulated bars or wire rope slings be used to simulate the insulators. If compression or cantilever insulators are planned for the transmission line, members that simulate those conditions should be used in the test.

Compression on unbraced panels caused by bridling of load lines should be avoided.

As a structure deflects under load, load lines may change their direction of pull. Adjustments should be made in the applied loads or the test rigging should be offset accordingly so that the vertical, transverse, and longitudinal vectors at the load points are the loads specified in the tower loading schedule.

C6.9 LOADING PROCEDURE

It is recommended that the primary consideration in establishing the sequence of testing should be that those load cases that have the least influence on the results of successive tests be tested first. A secondary consideration should be to simplify the operations necessary to carry out the test program.

C6.10 LOAD MEASUREMENT

The effects of pulley friction should be minimized. Load measurement by monitoring the load in a single part of a multipart block and tackle arrangement should be avoided. Accuracy of load measurement should be within 2% of the applied load.

C6.11 DEFLECTIONS

Points to be monitored should be selected to verify the deflections predicted by the design analysis. Accuracy of deflection measurements should be within the greater of 1 in. (25 mm) or 5% of the total deflection.

C6.13 DISPOSITION OF PROTOTYPE

An undamaged structure is usually accepted for use in the transmission line after all components are visually inspected and found to be structurally sound and within tolerances.

CHAPTER C7
STRUCTURAL MEMBERS AND CONNECTIONS USED IN FOUNDATIONS

C7.1 INTRODUCTION

The material in Chapter 7 covers structural members and connections normally supplied by the steel fabricators.

C7.2 GENERAL CONSIDERATIONS

C7.2.2 Pressed Plates For the pressed plate foundation, a relatively thick steel plate washer is welded to the bottom of the stub angle. The plate washer allows an attachment for bolting to the pressed plate and provides a transfer of the axial load from the stub angle to the corners of the pressed plate.

C7.3 DETERIORATION CONSIDERATIONS

Concrete foundations should be properly sloped to drain so that water pockets do not accumulate with ground material and cause excessive corrosion of the tower base material. If towers are located where groundwater can be highly corrosive, such as ash pits, industrial drainage areas, and oil refineries, concrete foundations should be used. If steel is exposed to such a groundwater environment, special protection is essential. Proper drainage around the steel should be established, and periodic inspections should be conducted. In some cases, it may be necessary to apply an additional protective coating, such as a bitumastic compound, to the steel. If new towers are located in such an environment, any steel members exposed to this severe ground condition should be increased in thickness at least 1/16 in. (1.6 mm) as a corrosion allowance.

C7.5 DESIGN REQUIREMENTS FOR CONCRETE AND REINFORCING STEEL

ACI 349 (1985) requirements for anchorage material have been modified for use in this standard. The formulas in this standard use yield strength, F_y, of the anchorage material and are suitable to ensure a ductile failure of the anchor material before a brittle failure of the concrete.

The ACI (1985) *Code Requirements for Nuclear Safety-Related Concrete Structures* recommend the following side cover distance for tension to prevent failure caused by lateral bursting for smooth anchor bolts.

$$m_t = 0.66d\sqrt{\frac{F_u}{\sqrt{f_c'}}} \qquad \text{(C7.5-1)}$$

Eq. (C7.5-1) can be used for stub angles if an equivalent diameter is determined based on the net section of the stub angle; m_t is measured from any portion of the stub angle to the nearest edge of the concrete.

ACI (1985) recommends the following side cover distance for anchorage shear:

$$m_v = 1.8d\sqrt{\frac{F_u}{\sqrt{f_c'}}} \qquad \text{(C7.5-2)}$$

For anchor bolts with base plates resting on concrete, shear is transmitted from the bolt at the surface forming a concrete wedge. Translation of the wedge under the shear force cannot occur without an upward thrust of the wedge on the base plate. This thrust induces a clamping force, and this mechanism is called shear friction. Therefore, the concrete or grout serves in developing the clamping action. Shear lugs can be used to transfer the shear load to the concrete and should be located in a concrete compression zone unless adequate reinforcing is provided. A combination of shear lugs and shear friction is not permitted.

In determining the side cover distance for shear, the anchor bolt diameter required for shear only can be used in Eq. (C7.5-2). However, this distance should not be less than $m_v/3$ determined for the actual d of the anchor bolt used or 4 inches.

Eq. (C7.5-2) can be used for stub angles if an equivalent diameter is determined based on the net section of the angle; m_v is measured from any portion of the stub angle to the nearest edge of the concrete.

Reinforcement as shown in Fig. C7-1 assists in containing tension and/or shear blowout. An additional reference is Shipp and Haninger (1983).

C7.6 SHEAR CONNECTORS

C7.6.1 Stud Shear Connectors [See Fig. 7-3(c)] Additional information is available on stud shear connectors in Salmon and Johnson (1990).

C7.6.2 Angle Shear Connectors [See Fig. 7-3(b)] The values obtained using Eq. (7.6-2) are based on initial yielding of the angle in bending at a stress of $1.19f'_c$ in the concrete. The 1.19 constant is based on the concrete pier having an area at least four times as large as the bearing area of a pair of shear connector angles.

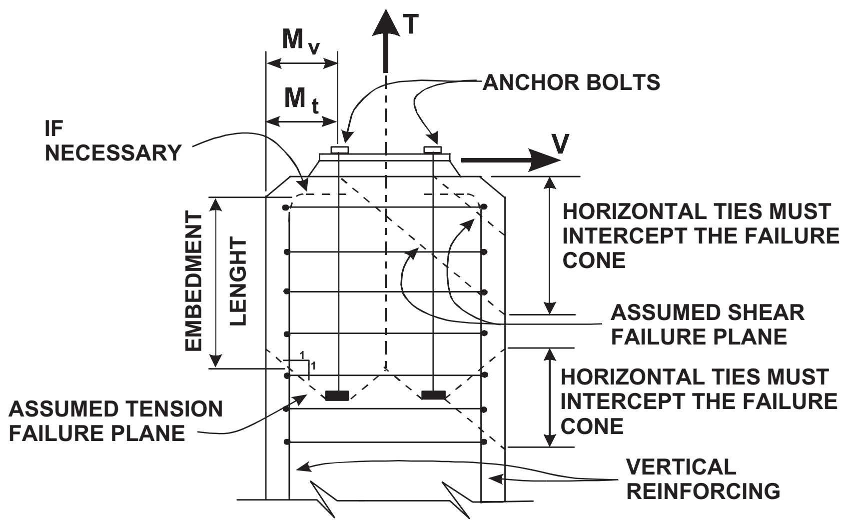


FIG. C7-1. Pier Reinforcing

CHAPTER C8
QUALITY ASSURANCE AND QUALITY CONTROL

C8.1 INTRODUCTION

A well planned and executed quality assurance (QA) and quality control (QC) program is necessary to ensure delivery of acceptable material in a timely manner. The objective of the program is to establish that transmission material is in conformance with the specifications of the purchase contract. A clear and concise contract between purchaser and supplier is an important part of the procedure necessary to obtain acceptable transmission towers. It is helpful if the responsibilities of both the purchaser and the supplier are defined in the contract so that no part of the process used to purchase, design, detail, test, fabricate, and deliver transmission material is omitted.

The purchaser's QA program outlines the methods, types of inspections, and records that are necessary to provide suitable production controls.

The supplier's QC program is a written document or series of information memoranda that establish procedures and methods of operation that affect the quality of the product. The supplier has complete control over the QC program and modifies it to adjust to changing requirements of a particular operation.

A joint review of the QA and QC requirements and agreement by both the purchaser and the supplier allows for any conflict resolution before the ordering of materials. Typical items discussed during the review process are the review and acceptance of the supplier's material specifications, sources of supply, material identification, storage, traceability procedures, and acceptance of certified mill test reports.

C8.2 QUALITY ASSURANCE

A quality assurance program documents the methods followed to establish appropriate review and interface with the supplier's quality control procedures. This program ensures that the contract can proceed smoothly, that proper communication channels are established with the responsible personnel to minimize confusion, that the purchaser's requirements are properly met, and that proper guidance and adequate technical support are supplied throughout the period of the contract.

The QA program defines the personnel who have the responsibility for contract performance, engineering, inspection, and receipt of material. Contract performance covers enforcement of the terms of the contract relative to payment, delivery commitments, contract changes, and legal matters. Engineering covers technical matters relative to design adequacy, detail requirements, production controls, structure testing, and shipment of material. Inspection covers the purchaser's material certification and production requirements. The QA program requires a close in-shop relationship with the supplier. In some instances, the purchaser uses contract personnel for this activity. Receipt of material by the purchaser's designated representative covers invoice certification that correct material has been received in the field.

As part of the QA program, the purchaser or a designated representative may inspect the supplier's equipment and facilities to ensure that the procedures followed during production meet the specific job requirements. The inspection may cover material certification, material handling, cutting and piece mark identification procedures, bending, welding, nondestructive testing required, galvanizing, fit-up requirements, and bundling for shipment.

C8.3 QUALITY CONTROL

The QC program establishes appropriate quality control checks for the supplier's planning and scheduling, engineering, drafting, purchasing, production, and shipping. Planning and scheduling are important to ensure that the work proceeds in an orderly manner. Engineering is responsible for specification, design, and shop drawing review in accordance with accepted standards and loading criteria. Drafting covers shop detail drawings that correctly and adequately interpret the design drawings and specifications. Purchasing secures appropriate materials and services in accordance with the requirements of the contract drawings and specifications. Production is responsible for all fabricating activities, receipt of material, storage, material preparation, processing, marking, welding, galvanizing, assembly, and shipping. The QC program establishes the procedures for determining that the finished product meets the purchaser's allowable tolerances.

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APPENDIX A
NOTATION

The following symbols are used in this Standard:

A = cross-sectional area (in.^{2}) (mm^2)
A_g = gross cross-sectional area (in.^{2}) (mm^2), or gross area of angle, or net area if there is a hole in the angle at the intersecting plane of the concrete foundation and the angle (in.^{2}) (mm^2)
A_n = net cross-sectional area (in.^{2}) (mm^2)
A_s = tensile stress area of bolt (in.^{2}) (mm^2)
A_{sc} = cross-sectional area of stud shear connector (in.^{2}) (mm^2)
A_t = minimum net area in tension (in.^{2}) (mm^2), or minimum net area in tension from the hole to the toe of the angle perpendicular to the line of force (in.^{2}) (mm^2)
A_v = minimum net area in shear (in.^{2}) (mm^2)
a = distance from shear center to load plane (in.) (mm)
b, b_1, b_2 = effective design widths of elements (in.) (mm), or width of leg $-t/2$ (in.) (mm)
C = constant based on ratio of f_1 and f_2
C_b = coefficient in formula for design bending stress
C_c = column slenderness ratio separating elastic and inelastic buckling
C_m = coefficient applied to bending term in interaction formula for prismatic members
C_w = warping constant of cross section (in.^{6}) (mm^6)
D = downthrust load; net difference in compression and uplift reactions on anchor bolts (kip) (N)
d = nominal diameter of bolt (in.) (mm), or minimum depth of stiffener (in.) (mm)
d_h = diameter of attachment hole (in.) (mm)
E = modulus of elasticity of steel (29,000 ksi) (200,000 MPa)
E_c = modulus of elasticity of concrete (ksi) (MPa)
e = distance from center of hole to end of member (in.) (mm)
e_m = required distance from center of hole to end of member (in.) (mm)
F_a = design axial compressive stress in prismatic member in absence of bending moment (ksi) (MPa)
F_b = design bending stress in prismatic member in absence of axial force (ksi) (MPa)
F_{cr} = critical stress for local buckling of plain angle members (ksi) (MPa)
F_t = design axial tensile stress (ksi) (MPa)
$F_{t(v)}$ = design axial tensile stress in conjunction with shear stress (ksi) (MPa)
F_u = specified minimum tensile strength (ksi) (MPa)
F_v = design shear stress (ksi) (MPa), or design average shear stress for beam webs (ksi) (MPa)
F_y = specified minimum yield stress (ksi) (MPa)
f = stress in compression element computed on the basis of effective design width (ksi) (MPa), or distance from center of hole to edge of member (in.) (mm)
f_1, f_2 = stress, in tension or compression, on an element (ksi) (MPa)
f'_c = specified compressive strength of concrete at 28 days (ksi) (MPa)
f_m = required distance from center of hole to edge of member (in.) (mm)
f_v = computed shear stress (ksi) (MPa)
g = transverse spacing locating fastener gauge lines (in.) (mm)
h = clear distance between flanges of beam (in.) (mm)
I = moment of inertia in truss plane (in.^{4}) (mm^4)
I_{ps} = polar moment of inertia about shear center (in.^{4}) (mm^4)
I_u = moment of inertia about U–U axis (in.^{4}) (mm^4)
I_x = moment of inertia about X–X axis (in.^{4}) (mm^4)
I_y = moment of inertia about Y–Y axis (in.^{4}) (mm^4)
I_z = moment of inertia about Z–Z axis (in.^{4}) (mm^4)
J = torsional constant of cross section (in.^{4}) (mm^4)
j = section property for torsional-flexural buckling (in.) (mm)
K = effective length factor for prismatic member
K_t = effective length factor for warping and rotation
K_u, K_x, K_y = effective length factor for buckling in designated axis
K' = effective length factor for a member with varying load and equal subpanel unsupported lengths
L = unbraced length of column (in.) (mm), distance from center of attachment hole to member edge (in.) (mm)

L_x, L_y = unbraced length in designated axis (in.) (mm)
L' = equivalent unsupported length of a member with varying load and equal length subpanels (in.) (mm)
M_{ax} = allowable bending moment about X–X axis (in.-kip) (mm-N)
M_{ay} = allowable bending moment about Y–Y axis (in.-kip) (mm-N)
M_b = lateral buckling moment for angles (in.-kip) (mm-N)
M_e = elastic critical moment (in.-kip) (mm-N)
M_x = bending moment about X–X axis (in.-kip) (mm-N)
M_y = bending moment about Y–Y axis (in.-kip) (mm-N)
M_{yc} = moment causing yield at extreme fiber in compression (in.-kip) (mm-N)
M_{yt} = moment causing yield at extreme fiber in tension (in.-kip) (mm-N)
M_1 = smaller moment at end of unbraced length of beam column (in.-kip) (mm-N)
M_2 = larger moment at end of unbraced length of beam column (in.-kip) (mm-N)
n = number of threads per unit of length (in.) (mm)
P = capacity of angle shear connector (kip) (N), axial tension or compression load on member (kip) (N), force transmitted by a bolt (kip) (N)
P_a = design axial compression load on member (kip) (N)
P_{ex} = Euler buckling load in X–X axis (kip) (N)
P_{ey} = Euler buckling load in Y–Y axis (kip) (N)
Q_n = capacity of a shear connector (kip) (N)
r = governing radius of gyration (in.) (mm)
r_{ps} = polar radius of gyration about shear center (in.) (mm)
r_t = equivalent radius of gyration for torsional buckling (in.) (mm)
r_{tf} = equivalent radius of gyration for torsional-flexural buckling (in.) (mm)
r_u = radius of gyration for U–U axis (in.) (mm)
r_x = radius of gyration for X–X axis (in.) (mm)
r_y = radius of gyration for Y–Y axis (in.) (mm)
r_z = radius of gyration for Z–Z axis (in.) (mm)
S_u, S_x, S_y, S_z = elastic section modulus in designated axis (in.^{3}) (mm^3)
S_{xc} = elastic section modulus about X–X axis of compression flange (in.^{3}) (mm^3)
s = longitudinal center-to-center spacing (pitch) of any two consecutive holes (in.) (mm)
s_m = required spacing between centers of adjacent holes (in.) (mm)
T = axial tensile load on anchor bolts (kip) (N)
t = thickness of element (in.) (mm)
u = U–U axis designation
u_o = distance between shear center and centroid (in.) (mm)
V = shear load perpendicular to anchor material or parallel to the intersecting plane (kip) (N)
V, V_1, V_2 = shear in a single-angle beam (kip) (N)
w = flat width of element (in.) (mm)
w_s = flat width of edge stiffener (in.) (mm)
x = X–X axis designation
y = Y–Y axis designation
y_o = distance between shear center and centroid (in.) (mm)
z = Z–Z axis designation
α = angle between bracing member and supported members (degrees)
θ = angle between flange and stiffener lip, angle between load and z-axis (degrees)
μ = coefficient of friction
ϕ = resistance factor
Ψ = unit factor as specified in text

APPENDIX B
EXAMPLES

The following examples have been included to illustrate many of the requirements of the Standard. These examples are not intended to represent all the requirements of the Standard and should not be used without first securing competent advice with respect to their suitability for any given application. As stated in Section 3.18, design values other than those illustrated in these examples may be used if substantiated by experimental or analytical investigation.

EXAMPLE 1. EQUAL LEG ANGLE WITH SYMMETRICAL BRACING

(1 in. = 25.4 mm; 1 ksi = 6.89 MPa; 1 kip = 4,450 N)

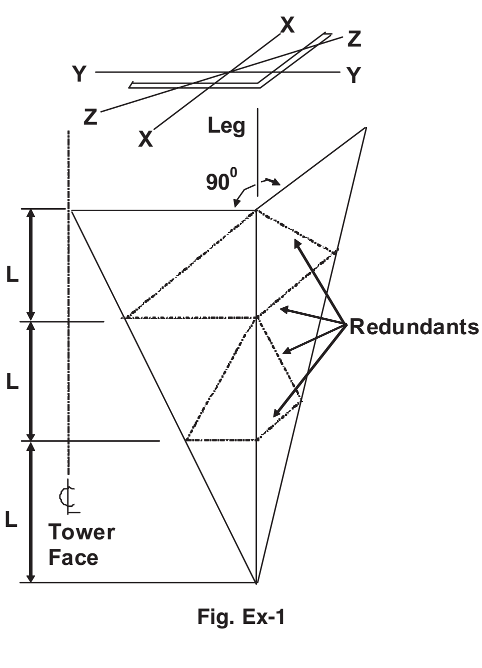


Fig. Ex-1

Leg Requirements:
L/r_z critical factor, Concentric loading, Eq. (3.7-4)
Eccentricities at leg splices should be minimized. Thicker leg sections should be properly butt-spliced.

This method of support minimizes rolling of the angle under load.

$L8 \times 8 \times 9/16$; $w/t = 12.1$; $r_z = 1.59$ in.; $r_x = r_y = 2.50$ in.; $A = 8.68$ in.^{2}

For 36 ksi yield steel; $w/t_{\lim} = 80/\sqrt{36} = 13.3$; L = 121 in.;

$$L/r_z = 121/1.59 = 76;\ F_y = 36 \text{ ksi.}$$

From Eq. (3.6-1), F_a = 29.5 ksi; Design strength = 29.5 × 8.68 = 256 kip.

$L8 \times 8 \times 9/16$; $w/t = 12.1$; $r_z = 1.59$ in.; $r_x = r_y = 2.50$ in.; $A = 8.68$ in.^{2}

For 36 ksi yield steel; $w/t_{\lim}$ = 13.3; L = 238 in.; L/r_z = 238/1.59 = 150; F_y = 36 ksi.
From Eq. (3.6-2), F_a = 12.7 ksi; Design strength = 12.7 × 8.68 = 110 kip.

$L6 \times 6 \times 5/16$; $w/t = 16.6$; $r_z = 1.20$ in.; $r_x = r_y = 1.89$ in.; $A = 3.65$ in.^{2}

For 36 ksi yield steel; $w/t_{\lim}$ = 13.3; F_{cr} = 30 ksi; L = 180 in.; L/r_z = 180/1.20 = 150.
From Eq. (3.6-2), F_a = 12.7 ksi; Design strength = 12.7 × 3.65 = 46.4 kip.

$L8 \times 8 \times 9/16$; $w/t = 12.1$; $r_z = 1.59$ in.; $r_x = r_y = 2.50$ in.; $A = 8.68$ in.^{2}

For 50 ksi yield steel; $w/t_{\lim}$ = 11.3; F_{cr} = 47.6 ksi; L = 121 in.; L/r_z = 76.
From Eq. (3.6-1), F_a = 35.6 ksi; Design strength = 35.6 × 8.68 = 309 kip.

$L8 \times 8 \times 9/16$ except $L = 238$ in.; $L/r_z = 150$.

From Eq. (3.6-2), F_a = 12.7 ksi; Design strength = 12.7 × 8.68 = 110 kip.

EXAMPLE 2. EFFECT OF END CONNECTIONS ON MEMBER CAPACITY

(1 in. = 25.4 mm; 1 ksi = 6.89 MPa; 1 kip = 4,450 N)

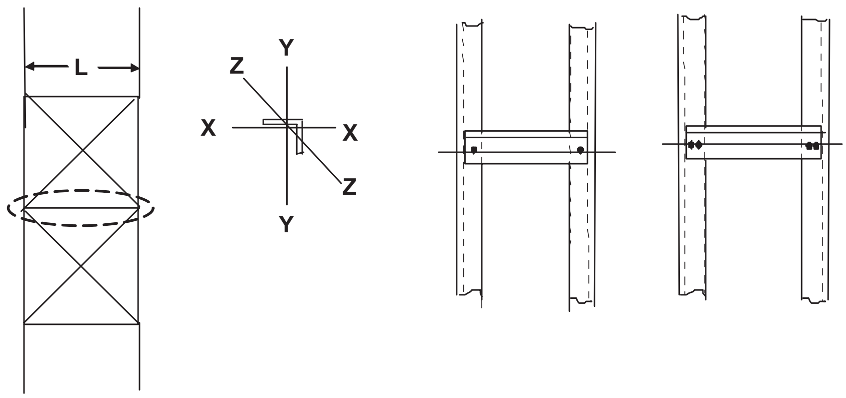


Fig. Ex-2

Tension-Only System with Compression Struts	Single Bolt Connection; No Restraint at Ends	Multiple Bolt Connection; Partial Restraint at Both Ends
L/r_z = Critical factor Eccentricity in critical axis L/r_z = from 0 to 120 [Eq. (3.7-7)]. L 1 3/4 × 1 1/4 × 3/16. r_z = 0.27 in.; A = 0.53 in.^{2} 36 ksi yield steel. L = 32 in.; L/r_z = 32/0.27 = 119; [Eqs. (3.7-7) and (3.6-1)] F_a = 19.8 ksi; Design strength = 19.8 × 0.53 = 10.5 kip.	L/r_z = from 120 to 250 [Eq. (3.7-8)]. L = 54 in.; L/r_z = 54/0.27 = 200 [Eqs. (3.7-8) and (3.6-2)]. F_a = 7.2 ksi; Design strength = 7.2 × 0.53 = 3.8 kip.	L/r_z = from 120 to 250 [Eq. (3.7-10)]. L = 54 in.; L/r_z = 200 [Eqs. (3.7-10) and (3.6-2)]. F_a = 10.0 ksi; Design strength = 10.0 × 0.53 = 5.3 kip.

EXAMPLE 3. CONCENTRIC LOADING, TWO ANGLE MEMBER

(1 in. = 25.4 mm; 1 ksi = 6.89 MPa; 1 kip = 4,450 N)

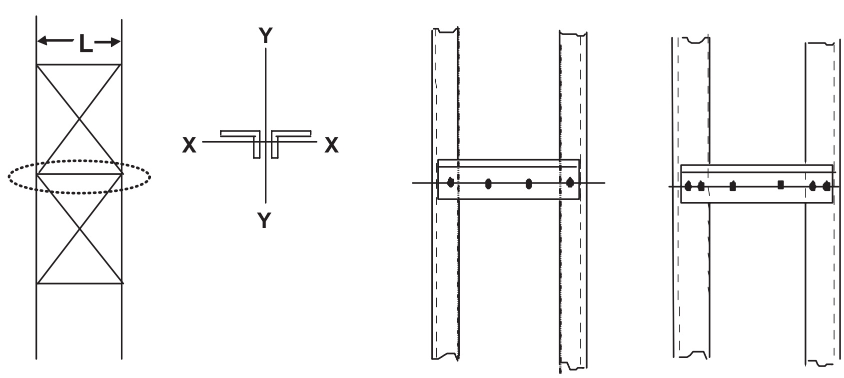


Fig. Ex-3

Tension-Only System with Compression Struts	Single Bolt Connections; No Restraint at Ends	Multiple Bolt Connections; Partial Restraint at Both Ends
L/r_x or L/r_y = Critical factor concentric loading. L/r_x or L/r_y = from 0 to 120 [Eq. (3.7-5)]	L/r_x or L/r_y = from 120 to 200 [Eq. (3.7-8)]	L/r_x or L/r_y = from 120 to 250 [Eq. (3.7-10)]

EXAMPLE 4. K-BRACING, TWO ANGLE MEMBER

(1 in. = 25.4 mm; 1 ksi = 6.89 MPa; 1 kip = 4,450 N)

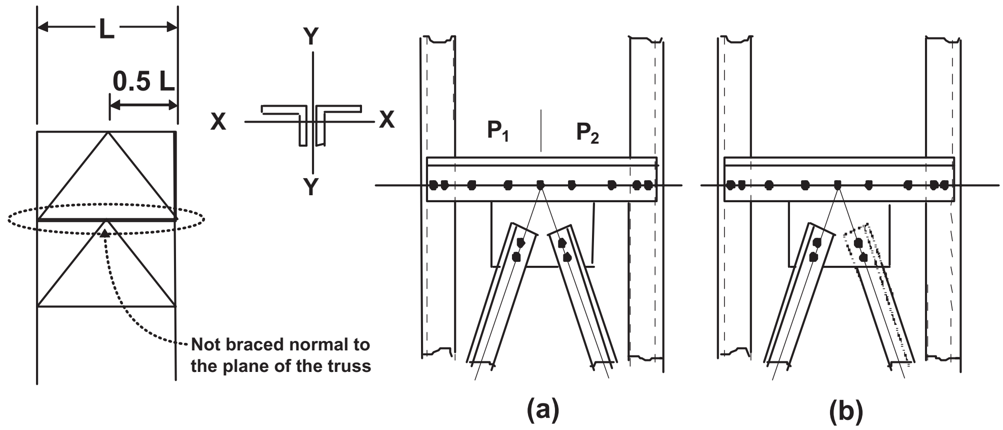


Fig. Ex-4

	Bracing Requirements	
Tension–Compression System with Compression Struts.	Concentric Load at Ends, Eccentric Loading at Intermediate in Both Directions (a).	Concentric Loading at Ends and Intermediate (b).

Constant Load ($P_1 = P_2$) in Double Angle

Multiple bolt connections		
$0.5L/r_x$ or L/r_y critical factor. Partial restraint at one end. $0.5L/r_x$ from 120 to 225 [Eq. (3.7-9)]. See Statement in Section C3.7.4(c). Partial restraint at both ends. L/r_y from 120 to 250 [Eq. (3.7-10)].	$0.5L/r_x$ [Eq. (3.7-6)] or L/r_y [Eq. (3.7-5)] from 0 to 120.	$0.5L/r_x$ or L/r_y from 0 to 120 [Eq. (3.7-5)].

Varying Load (P_1 not equal to P_2) in Equal Length Subpanel of Double Angle Check member for forces P_1 and P_2 for in-plane bucking using $0.5L/r_x$, as given previously.

Check out-of-plane buckling on the y–y axis of the angle over the length L, as follows.

$$KL/r_y = K(K'L/r_y)$$

where K' shall be determined from:

1. When P_1 and P_2 are compression forces, use Eq. (3.7-14a).

   $$K' = 0.75 + 0.25\,(P_2/P_1),\ \text{where } P_1 > P_2$$

   Sample Calculation: Assuming that $P_1 = -5.6$ kip, $P_2 = -3.3$ kip and concentric loads

   $$K' = 0.75 + 0.25\,(3.3/5.6) = 0.90.$$

2. When either P_1 or P_2 is a tension force, use Eq. (3.7-14b)

   $$K' = 0.75 - 0.25\,(P_2/P_1),\ \text{where } P_2 \text{ is the tension force}$$

   Sample Calculation: Assuming that $P_1 = -44.6$ kip, $P_2 = 2.2$ kip and concentric loads. Note: The negative sign in Eq. (3.7-14b) accounts for the different (–) compression and (+) tension forces. Therefore, the absolute magnitude of the forces shall be substituted into the equation.

   $$K' = 0.75 - 0.25\,(2.2/44.6) = 0.74.$$

The following example is given to demonstrate the situation when eccentric loading exists. Therefore, the preceding figure does not represent the following example.

For members with eccentric loading, the K' previously determined by Eq. (3.7-14b) should be adjusted using Eq. (3.7-7).

Sample Calculation: Assuming 2. above (when either P_1 or P_2 is a tension force) and changing the double angle to a single angle with normal framing eccentricity at both ends, $L = 9.0$ ft, $r_y = 1.38$.

Equivalent unsupported length = $K'L = L' = (0.74)(9.0) = 6.66$ ft; $L'/r_y = (6.66)(12)/(1.38) = 57.9$.

Using Eq. (3.7-7), $KL'/r_y = 60 + 0.5(57.9) = 89$.

EXAMPLE 5. EFFECT OF SUBDIVIDED PANELS AND END CONNECTIONS

(1 in. = 25.4 mm; 1 ksi = 6.89 MPa; 1 kip = 4,450 N)

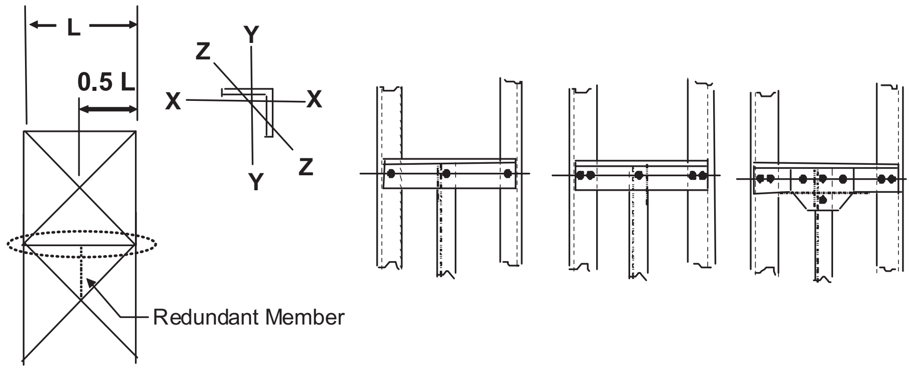


Fig. Ex-5

Tension-Only System with Compression Struts	Single Bolt Connection; No Restraint at Ends or Intermediate	Multiple Bolt Connection at Ends; Single Bolt Connection at Intermediate Point. Partial Restraint at One End. No Restraint at Intermediate	Multiple Bolt Connection; Partial Restraint at Ends and Intermediate
		See statement in Section C3.7.4 concerning partial restraint.	
$0.5L/r_z$ or L/r_y critical factor. Eccentricity in critical axis. $0.5L/r_z$ [Eq. (3.7-6)] or L/r_y [Eq. (3.7-7)] from 0 to 120.	$0.5L/r_{zz}$ or L/r_y from 120 to 200 [Eq. (3.7-8)].	$0.5L/r_z$ from 120 to 225 [Eq. (3.7-9)].	$0.5L/r_z$ or L/r_y from 120 to 250 [Eq. (3.7-10)].
		Partial restraint at both ends.	
$L1\ 3/4 \times 1\ 1/4 \times 3/16$ $r_z = 0.27$ in.; $r_y = 0.37$ in. $A = 0.53$ in.^{2} 36 ksi yield steel. L/r_y, critical: $L = 44$ in.; $L/r_y = 44/0.37 = 119$; [Eqs. (3.7-7) and (3.6-1)]. $F_a = 19.8$ ksi. Design strength = $19.8 \times 0.53 = 10.5$ kip.	$L/r_y = 74/0.37 = 200$ [Eqs. (3.7-8) and (3.6-2)]. Design strength = $7.2 \times 0.53 = 3.8$ kip.	L/r_y from 120 to 250 [Eq. (3.7-10)]. $L/r_y = 200$ [Eqs. (3.7-10) and (3.6-2)]. Design strength =$10.0 \times 0.53 = 5.3$ kip.	$L/r_y = 200$ [Eqs. (3.7-10) and (3.6-2)]. Design strength = $10.0 \times 0.53 = 5.3$ kip.

EXAMPLE 6. CONCENTRIC LOADING, TWO ANGLE MEMBER, SUBDIVIDED PANELS

(1 in. = 25.4 mm; 1 ksi = 6.89 MPa; 1 kip = 4,450 N)

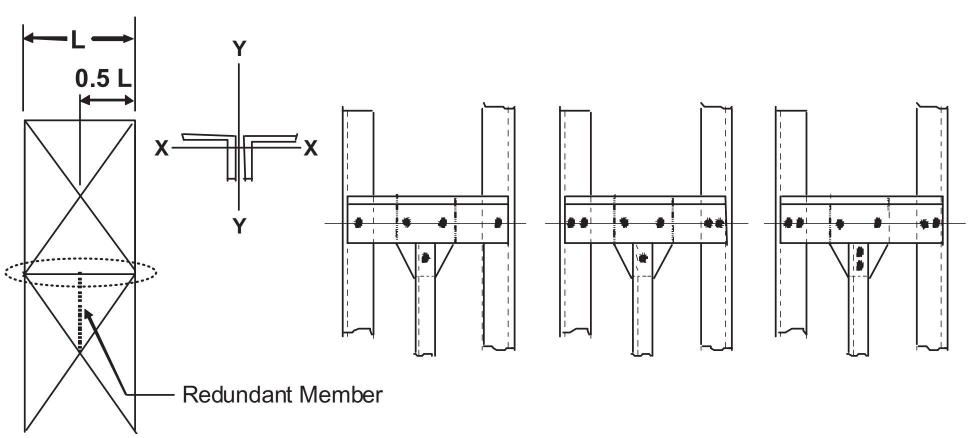


Fig. Ex-6

Tension-Only System with Compression Struts	Bracing Requirements: Single Bolt Connection; No Restraint at Ends or Intermediate	Multiple Bolt Connection at Ends; Single Bolt Connection at Intermediate Point. Partial Restraint at One End. No Restraint at Intermediate	Multiple Bolt Connection; Partial Restraint at Ends and Intermediate
		See statement in Section C3.7.4 concerning partial restraint.	
$0.5L/r_x$ or L/r_y critical factor. Concentric loading $0.5L/r_x$ or L/r_y from 0 to 120 [Eq. (3.7-5)].	$0.5L/r_x$ or L/r_y from 120 to 200 [Eq. (3.7-8)].	$0.5L/r_x$ from 120 to 225 [Eq. (3.7-9)].	$0.5L/r_x$ or L/r_y from 120 to 250 [Eq. (3.7-10)].
		Partial restraint at both ends. L/r_y from 120 to 250 [Eq. (3.7-10)].	

EXAMPLE 7. X-BRACE SYSTEMS WITH NO INTERMEDIATE REDUNDANT SUPPORTS

(1 in. = 25.4 mm; 1 ksi = 6.89 MPa; 1 kip = 4,450 N)

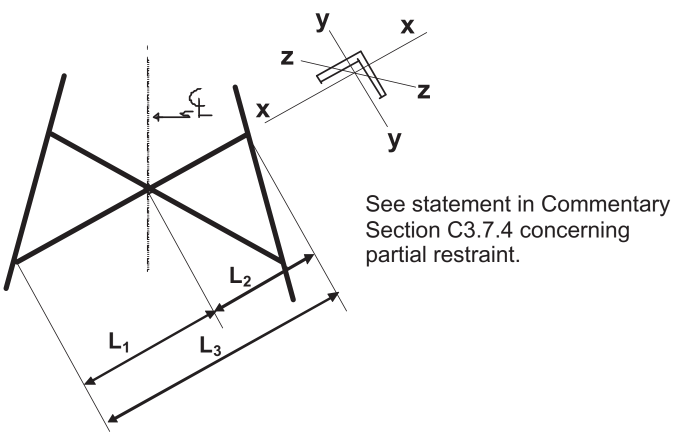


Fig. Ex-7

1. Tension/compression system with members connected at the crossover point. If the member in tension has a force of not less than 20% of the force in the compression member, the crossover point provides support resisting out-of-plane buckling.

$$L_1 \geq L_2$$

For L/r from 0 to 120.
Critical factor L_1/r_z. Use Eq. (3.7-6).

For L/r from 120 to 200.
Critical factor L_1/r_z.
For single bolt connections at the post leg, use Eq. (3.7-8).
For multiple bolt connections at the post leg, use Eq. (3.7-9).
Providing end restraint can be assumed at the post leg; otherwise use Eq. (3.7-8).

Note: When unequal legs are used and the long leg is connected, the critical factor for out-of-plane buckling should be $(L_1 + 0.5L_2)/r_y$ using the appropriate equation for KL/r, as given previously.

2. Tension/compression system with members connected at the crossover point. If the member in tension has a force of less than 20% of the force in the compression member, or if both members in the same panel are in compression, the crossover point does not provide support resisting out-of-plane buckling.

$$L_1 \geq L_2$$

For L/r from 0 to 120.
Critical factor L_1/r_z. Use Eq. (3.7-6).
Critical factor $L_3/r_{x \text{ or } y}$. Use Eq. (3.7-7).

For L/r from 120 to 200.
(a) Critical factor L_1/r_z.
For single bolt connections at the post leg, use Eq. (3.7-8).
For multiple bolt connections at the post leg, use Eq. (3.7-9).
Providing end restraint can be assumed at the post leg; otherwise use Eq. (3.7-8).
(b) Critical factor $L_3/r_{x \text{ or } y}$.
For single bolt connections at the post leg, use Eq. (3.7-8).
For multiple bolt connections at the post leg, use Eq. (3.7-10).
Providing end restraint can be assumed at the post leg; otherwise use Eq. (3.7-8).

Note: When unequal legs are used and the long leg is connected, the critical factor for out-of-plane buckling should be L_3/r_y using the appropriate equation for KL/r, as given previously.

EXAMPLE 8. X-BRACE SYSTEMS WITH INTERMEDIATE REDUNDANT SUPPORTS—CASE 1

(1 in. = 25.4 mm; 1 ksi = 6.89 MPa; 1 kip = 4,450 N)

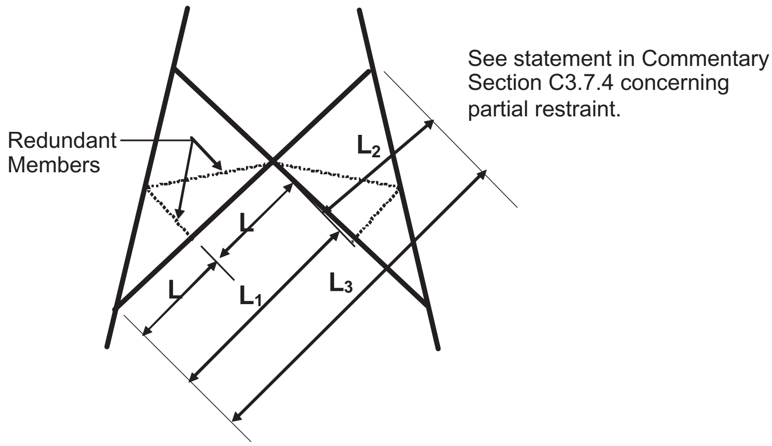


Fig. Ex-8

1. Tension/compression system with members connected at the crossover point. If the member in tension has a force of not less than 20% of the force in the compression member, the crossover point provides support resisting out-of-plane buckling.

$$L_1 > L_2$$

For L/r from 0 to 120.
Critical factor L/r_z or L_2/r_z, whichever is maximum. Use Eq. (3.7-6).
Critical factor $L_1/r_{x \text{ or } y}$. Use Eq. (3.7-6).

For L/r from 120 to 200.
(a) Critical factor L/r_z. Use Eq. (3.7-8); or L_2/r_z using Eq. (3.7-9) if end restraint can be assumed; otherwise use Eq. (3.7-8).
(b) Critical factor $L_1/r_{x \text{ or } y}$.
For single bolt connections, use Eq. (3.7-8).
For multiple bolt connection where end restraint can be assumed at one end, use Eq. (3.7-9); otherwise use Eq. (3.7-8).

No end restraint is assumed in segment between crossover point and intermediate redundant support point.

Note: When unequal legs are used and the long leg is connected, the critical factor for out-of-plane buckling should be $(L_1 + 0.5L_2)/r_y$ using the appropriate equation for KL/r, as given previously.

2. Tension/compression system with members connected at the crossover point. If the member in tension has a force of less than 20% of the force in the compression member, or if both members at the same panel are in compression, the crossover does not provide support resisting out-of-plane buckling.

$$L_1 > L_2$$

For L/r from 0 to 120.
Critical factor L/r_z or L_2/r_z, whichever is maximum. Use Eq. (3.7-6).
Critical factor $L_3/r_{x \text{ or } y}$. Use Eq. (3.7-7).

For L/r from 120 to 200;
(a) Critical factor L/r_z. Use Eq. (3.7-8); or L_2/r_z using Eq. (3.7-9) if end restraint can be assumed; otherwise use Eq. (3.7-8).
(b) Critical factor $L_3/r_{x \text{ or } y}$.
For single bolt connections at the post legs, use Eq. (3.7-8).
For multiple bolt connections at the post legs, use Eq. (3.7-10) if end restraint can be assumed at the post leg; otherwise use Eq. (3.7-8).

No end restraint is assumed in segment between crossover point and intermediate redundant support point.

EXAMPLE 9. X-BRACE SYSTEMS WITH INTERMEDIATE REDUNDANT SUPPORTS—CASE 2

(1 in. = 25.4 mm; 1 ksi = 6.89 MPa; 1 kip = 4,450 N)

1. Tension/compression system with members connected at the crossover point. If the member in tension has a force of not less than 20% of the force in the compression member, the crossover point provides support resisting out-of-plane buckling.

$$L_1 \geq L_2 \text{ and } L > L_0$$

For L/r from 0 to 120.
Critical factor L/r_z. Use Eq. (3.7-6).
Critical factor $L_1/r_{x \text{ or } y}$. Use Eq. (3.7-6).

For L/r from 120 to 200.
(a) Critical factor L/r_z. Use Eq. (3.7-8).
(b) Critical factor $L_1/r_{x \text{ or } y}$.
For single bolt connections. Use Eq. (3.7-8).
For multiple bolt connection where end restraint can be assumed at one end, use Eq. (3.7-9); otherwise use Eq. (3.7-8).

No end restraint is assumed in segment between crossover point and intermediate redundant support point.

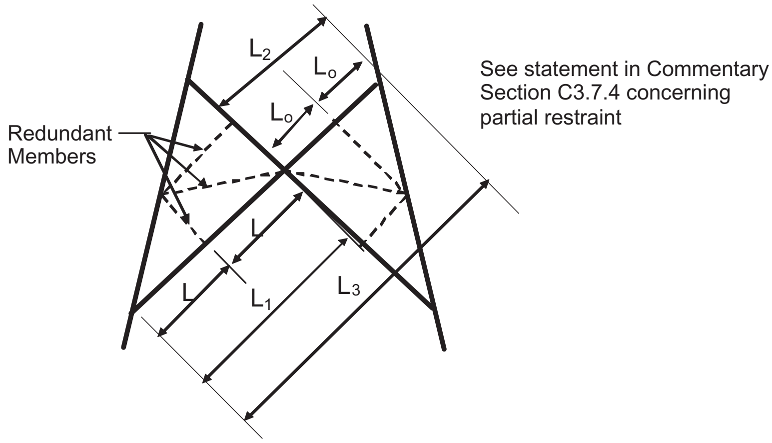


Fig. Ex-9

Note: When unequal legs are used and the long leg is connected, the critical factor for out-of-plane buckling should be $(L_1 + 0.5L_2)/r_y$ using the appropriate equation for KL/r, as given previously.

2. Tension/compression system with members connected at the crossover point. If the member in tension has a force of less than 20% of the force in the compression member, or if both members at the same panel are in compression, the crossover does not provide support resisting out-of-plane buckling.

$$L_1 > L_2 \text{ and } L > L_0$$

For L/r from 0 to 120.

Critical factor L/r_z or $L_1/r_{x\text{ or }y}$, whichever is maximum. Use Eq. (3.7-6).

Critical factor $L_3/r_{x\text{ or }y}$. Use Eq. (3.7-7).

For L/r from 120 to 200.

(a) Critical factor L/r_z. Use Eq. (3.7-8).

(b) Critical factor $L_3/r_{x\text{ or }y}$.

For single bolt connections. Use Eq. (3.7-8).

For multiple bolt connection at the post legs, use Eq. (3.7-10) if end restraint can be assumed at the post leg; otherwise use Eq. (3.7-8).

No end restraint is assumed in the segment between the crossover point and intermediate redundant support point.

Note: When unequal legs are used and the long leg is connected, the critical factor for out-of-plane buckling should be L_3/r_y using the appropriate equation for KL/r, as given previously.

EXAMPLE 10. COLD-FORMED ANGLE

(1 in. = 25.4 mm; 1 ksi = 6.89 MPa; 1 kip = 4,450 N)

Tension-Only System with Compression Strut

$$F_y = 50\text{ ksi.}$$

$$C_c = 106.9;\ r_z = 0.586\text{ in.; Area} = 0.777\text{ in.}^2,\ L = 117\text{ in.}$$

$$w/t = 2.6775/0.135 = 19.8;\ w/t_{\lim} = 80/\sqrt{F_y} = 11.3;$$

$$w/t = 144/\sqrt{F_y} = 20.4;\ 11.3 < 19.8 < 20.4.$$

Then $F_{cr} = (1.677 - 0.677 \times 19.8/11.3)50 = 24.6$ ksi; and

$$C_c = \pi\sqrt{2E/F_{cr}} = 3.14\sqrt{\frac{2(29{,}000)}{24.6}} = 152.5.$$

For a single bolt connection,

$$L/r_z = 117/0.586 = 200 > C_c$$

$$K = 1\text{ [Eq. (3.7-8)]; Design strength} = F_a \times A$$

$$= \frac{286{,}000}{(200)^2} \times 0.777 = 5.6\text{ kip.}$$

For a two bolt connection,

$$L/r_z = 200 > C_c,\text{ use Eq. (3.7-10).}$$

See statement in Section C3.7.4 concerning partial restraint.

$$KL/r = 46.2 + 0.615 \times 200 = 169.2.$$

$$\text{Design strength} = F_a \times A = \frac{286{,}000}{(169.2)^2} \times 0.777 = 7.8\text{ kip.}$$

Change unsupported length to 4 ft, 10 1/2 in.;

$$L/r_z = \frac{58.5}{0.586} = 100.$$

For a two bolt connection,

$$L/r_z < C_c\text{: eccentric connection, both ends use Eq. (3.7-7).}$$

$$KL/r = 60 + 0.5 \times 100 = 110;$$

$$F_a = \left[1 - 0.5\left(\frac{110}{152.5}\right)^2\right]24.6 = 18.2\text{ ksi.}$$

$$\text{Design strength} = F_a \times A = 18.2 \times 0.777 = 14.1\text{ kip.}$$

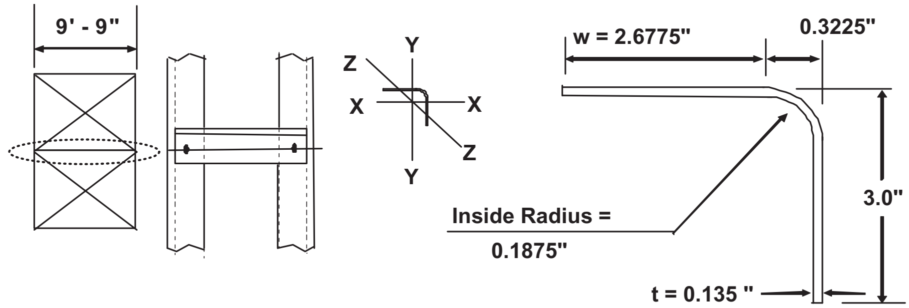


Fig. Ex-10

EXAMPLE 11. COLD-FORMED LIPPED ANGLE

(1 in. = 25.4 mm; 1 ksi = 6.89 MPa; 1 kip = 4,450 N)

Tension-Only System with Compression Strut

$$F_y = 50\text{ ksi}, C_c = 106.9, \left(\frac{w}{t}\right)\lim = \frac{220}{\sqrt{F_y}} = \frac{220}{\sqrt{50}} = 31.1$$

$$\frac{w}{t} = \frac{2.33}{0.135} = 17.3 < 31.1$$

where $\sin\theta = 1$

$$d_{\min} = 2.8t\sqrt[6]{\left(\frac{w}{t}\right)^2 - \frac{4000}{F_y}}$$

$$= 2.8 \times 0.135\sqrt[6]{(17.3)^2 - \frac{4000}{50}} = 0.93\text{ in.} < d$$

$A = 1.03\text{ in.}^2$; $C_w = 0.441\text{ in.}^6$; $\bar{u} = 1.34$ in.; $I_u = 1.79\text{ in.}^4$; $r_u = 1.32$ in.; $m = 0.30$ in.; $I_z = 0.616\text{ in.}^4$; $r_z = 0.773$ in.; $u_o = 1.34 + 0.30 = 1.64$ in.; $I_{ps} = 1.79 + 0.616 + 1.03 \times 1.64^2 = 5.18\text{ in.}^4$; $r_{ps} = 2.24$ in.; $J = 0.00623\text{ in.}^4$

$$r_t = \sqrt{\frac{0.441 + (0.04)(0.00623)(90)^2}{5.18}} = 0.689\text{ in.}$$

$$\frac{2}{r_{tf}^2} = \frac{1}{0.689^2} + \frac{1}{1.32^2}$$

$$+ \sqrt{\left(\frac{1}{0.689^2} - \frac{1}{1.32^2}\right)^2 + 4\left(\frac{1.64}{(0.689)(1.32)(2.24)}\right)^2} = 4.903$$

$$r_{tf} = \sqrt{\frac{2}{4.903}} = 0.639\text{ in.} < r_{zz}\ \frac{L}{r_{tf}} = \frac{90}{0.639} = 141$$

[two bolt connection; Eq. (3.7-10)]

$$\frac{KL}{r_{tf}} = 46.2 + 0.615 \times 141 = 133 \quad F_a = \frac{286{,}000}{(133)^2} = 16.2\text{ ksi.}$$

$$\text{Design strength} = F_a \times A = 16.2 \times 1.03 = 16.7\text{ kip.}$$

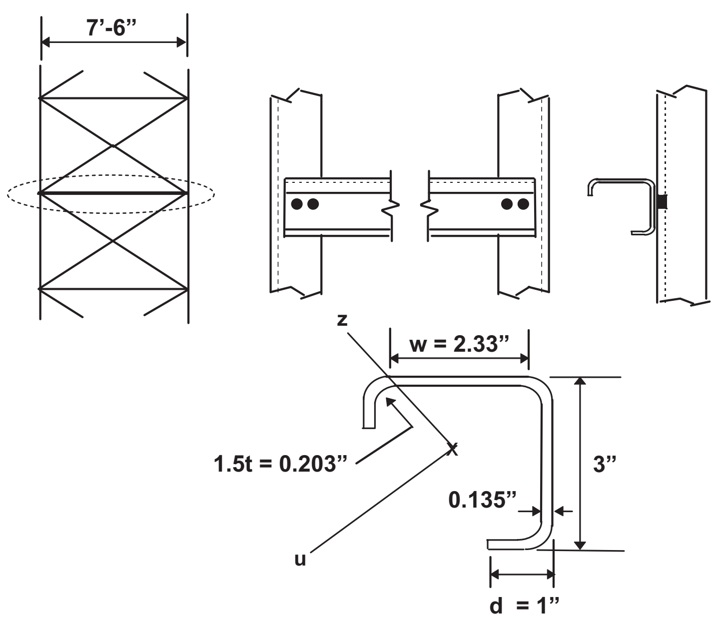


Fig. Ex-11

EXAMPLE 12. M-SECTION AS COLUMN MEMBER

(1 in. = 25.4 mm; 1 ksi = 6.89 MPa; 1 kip = 4,450 N)

M4 × 13; F_y = 50 ksi; C_c = 106.9; A = 3.81 in.^{2}; r_y = 0.939 in.; r_x = 1.66 in.; J = 0.19 in.^{4}; u_o = 0; I_x = 10.5 in.^{4}; I_y = 3.36 in.^{4}; S_x = 5.24 in.^{3}; S_y = 1.71 in.^{3}; K = 1; I_{ps} = 13.86 in.^{4}; C_w = 11.06

$$\text{Flange } w/t = 1.57/0.371 = 4.23$$
$$\text{Allowable } w/t = 72/\sqrt{50} = 10.2$$

$$\text{Web } w/t = 2.38/0.254 = 9.4$$
$$\text{Allowable } w/t = 220/\sqrt{50} = 31.1$$

Local buckling of elements is not critical.

1. Determine column design strength, concentrically loaded; L = 96 in., K_t = 1

$$\text{Eq. (3.8-2); } r_t = \sqrt{\frac{11.06 + (0.04)(0.19)(96)^2}{13.86}} = 2.419 \text{ in.}$$

$r_y = 0.939$ in.; therefore r_y controls

$$L/r_y = 96/0.939 = 102.2 < C_c$$

$$\text{therefore,} \quad F_a = \left[1 - 0.5\left(\frac{102.2}{106.9}\right)^2\right] 50 = 27.15 \text{ ksi.}$$

$$\text{Design strength} = F_a \times A = 27.15 \times 3.81 = 103.4 \text{ kip.}$$

2. Determine if suitable for 50-kip load if member is connected on one flange at each end (eccentricity: 2.0 in.).

   Use Eq. (3.12-1); P_a = 103.4 kip (prior calculations); K_x = 1; L = 96 in.; K_y = 1

$$P_{ex} = \frac{\pi^2 El_x}{(K_x L_x)^2} = \frac{3.14^2(29{,}000)(10.5)}{96^2} = 326$$

$M_1 = M_2$ end moments and single curvature;
$C_b = 1.75 + 1.05(-1) + (-0.3) = 1.0$

$$\text{Eq. (3.14-1);} \quad r^2 = \frac{1.0\sqrt{3.36}}{5.24}\sqrt{11.06 + (0.04)(0.19)(96^2)}$$
$$= 3.15;\ r = 1.775 \text{ in.}$$

$$L/r = 96/1.775 = 54;\ F_a = \left[1 - 0.5\left(\frac{54}{106.9}\right)^2\right] 50 = 43.6 \text{ ksi.}$$

$$M_{ax} = F_a \times S_x = 43.6 \times 5.24 = 228.5 \text{ in.-kip; } M_{ay} = 0$$

$$\text{Eq. (3.12-1); } \frac{50}{103.4} + \frac{50 \times 2}{228.5} \times \frac{1}{1 - 50/326}$$
$$= 0.484 + 0.515 = 0.999 < 1.0$$

Design strength of member is 50 kip. (This example illustrates the importance of minimizing eccentricities in end connections.)

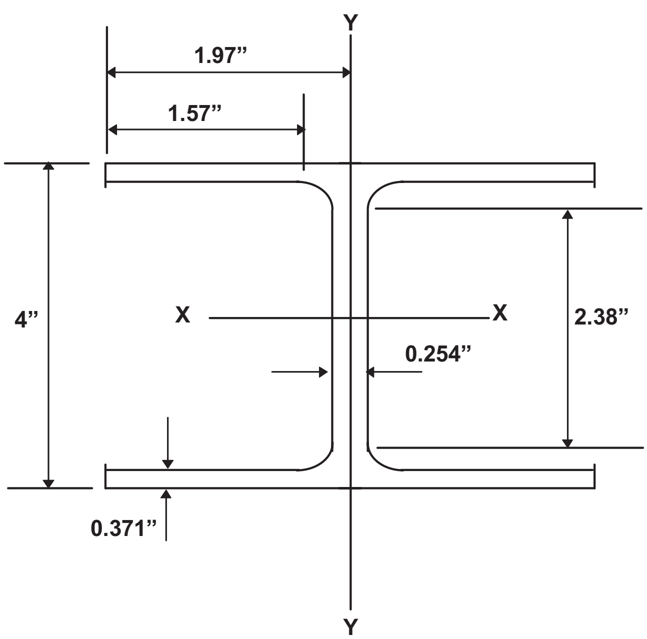


Fig. Ex-12

EXAMPLE 13. CHANNEL AS COLUMN

(1 in. = 25.4 mm; 1 ksi = 6.89 MPa; 1 kip = 4,450 N)

I_x = 4.34 in.^{4}; $\bar{x} = 0.354$ in.; l_y = 0.239 in.^{4}; m = 0.493 in.; $x_o = -(\bar{x} + m) = -0.847$ in.; $J = 0.004$ in.^{4}; $C_w = 1.37$; $r_x = 2.06$ in.; r_y = 0.48 in.; r_{ps} = 2.28 in.; I_{ps} = 5.3 in.^{4}; A = 1.02 in.^{2}; K = 1; F_y = 36 ksi; C_c = 126; L = 48 in.

1. Determine column design strength for concentric load. Note: X–X is axis of symmetry.

$$\text{Eq. (3.8-2);} \quad r_t = \sqrt{\frac{1.37 + (0.04)(0.004)(48)^2}{5.3}} = 0.57$$

Eq. (3.8-1); terms use $U-U$ as symmetrical axis. (Use $X-X$ values in place of $U-U$ values).

$$\frac{2}{r_{tf}^2} = \frac{1}{0.57^2} + \frac{1}{2.06^2}$$
$$+ \sqrt{\left(\frac{1}{0.57^2} - \frac{1}{2.06^2}\right)^2 + 4\left(\frac{0.847}{(0.57)(2.06)(2.28)}\right)^2}$$

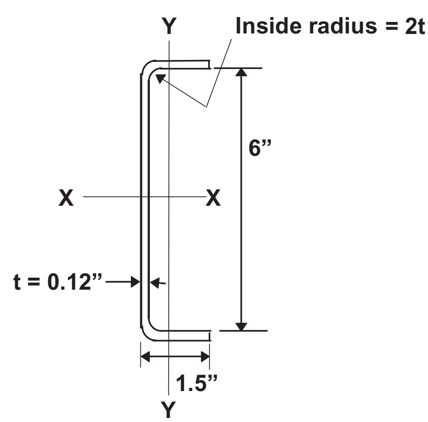


Fig. Ex-13

$$\frac{2}{r_{tf}^2} = 6.23; r_{tf} = 0.57$$

$$r_y < r_x \text{ or } r_{tf} L / r_y = 48/0.48 = 100;$$

$$F_a = \left[1 - 0.5\left(\frac{100}{126}\right)^2\right] 36 = 24.7 \text{ ksi} = f$$

Check flanges w/t = 1.14/0.12 = 9.5; Eq. (3.9-1) $= 72/\sqrt{f} = 72/\sqrt{24.7} = 14.5$

Check web w/t = 5.28/0.12 = 44; Eq. (3.9-3) $= 220/\sqrt{f} = 220/\sqrt{24.7} = 44.3$

Elements are fully effective. Design strength = $F_a \times A$ = 24.7 × 1.02 = 25.2 kip.

2. Determine if channel is suitable for 8.7 kip load if connection is to back of web.

End moments M_1 and M_2 are equal: C_m = 1.0;

Eccentricity about Y–Y axis = e = 0.354 + 0.06 = 0.41;

$$S_{my} = 0.239/1.09 = 0.22 \text{ (toe of flange)}$$

$$M_{ay} = 36 \times 0.22 = 7.9 \text{ in. kip;}$$

$$P_{ey} = \pi^2 EI_y / (K_y L_y)^2 = \frac{(3.14)^2 (29{,}000)(0.239)}{48^2}$$

$$M_{ax} = 0; P_{ey} = 29.7$$

$$\text{Eq. (3.12-1);}\ \frac{8.7}{25.2} + \frac{(8.7)(0.41)}{7.9}\left(\frac{1}{1 - 8.7/29.7}\right) = 0.35 + 0.64 = 0.99 < 1.0$$

Design strength of member is 8.7 kip. (This example illustrates the importance of minimizing eccentricities at the end connections).

EXAMPLE 14. T-SECTION AS COLUMN

(1 in. = 25.4 mm; 1 ksi = 6.89 MPa; 1 kip = 4,450 N)

u_o = –1.178 in.; I_y = 6.282 in.^{4}; r_y = 1.504 in.; I_x = 6.168 in.^{4}; r_x = 1.491 in.; r_{ps} = 2.423 in.; A = 2.775 in.^{2}; F_y = 50 ksi; C_c = 106.9; C_w = 13.567; J = 0.0327 in.^{4}; I_{ps} = 16.3 in.^{4}; L = 120 in.; K = 1; e = 1.412 in.

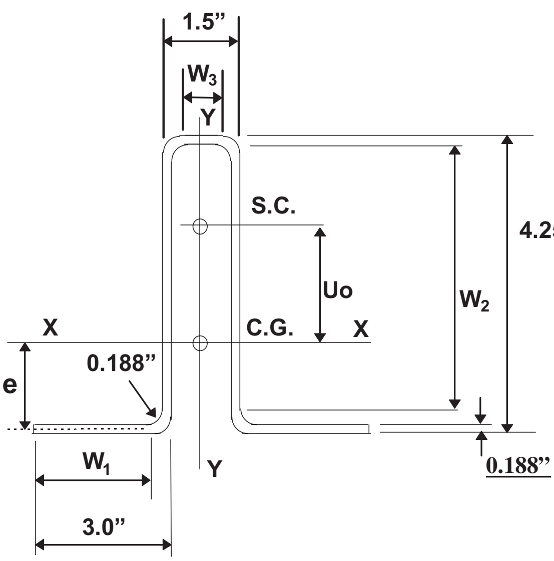


Fig. Ex-14

1. Determine design strength of concentric load.

$$\text{Eq. (3.8-2);}\quad r_t = \sqrt{\frac{13.567 + (0.04)(0.0327)(120)^2}{16.3}} = 1.41 \text{ in.}$$

Eq. (3.8-1); Terms use U–U as symmetrical axis. (Use Y–Y values in place of U–U values).

$$\frac{2}{r_{tf}^2} = \frac{1}{1.41^2} + \frac{1}{1.504^2} + \sqrt{\left(\frac{1}{1.41^2} - \frac{1}{1.504^2}\right)^2 + 4\left(\frac{-1.178}{(1.41)(1.504)(2.42)}\right)^2}$$

$$\frac{2}{r_{tf}^2} = 1.407 \quad r_{tf} = 1.19 < r_x$$

$$L/r_{tf} = 120/1.19 = 100.8; \text{Eq. (3.6-1).}$$

$$F_a = \left[1 - 0.5\left(\frac{100.8}{106.9}\right)^2\right] 50 = 27.7 \text{ ksi} = f$$

w_2/t = 18.6	Eq. (3.9-3)	$220/\sqrt{27.7} = 41.8 > 18.6$
w_3/t = 3.98	Eq. (3.9-3)	$220/\sqrt{27.7} = 41.8 > 3.98$
w_1/t = 13.96	Eq. (3.9-1)	$72/\sqrt{27.7} = 13.68 < 13.96$

Maximum for w_1 = 13.68 × 0.188 = 2.572 in. (w_1 is 2.624 in.).
New area = 2.775 – 2 × 0.188 (2.624 – 2.572) = 2.775 – 0.020 = 2.755 in.^{2}
Design strength = $F_a \times A$ = 27.7 ksi × 2.755 in.^{2} = 76.3 kip.

EXAMPLE 15. SCHIFFLERIZED ANGLE WITH SYMMETRICAL BRACING

(1 in. = 25.4 mm; 1 ksi = 6.89 MPa; 1 kip = 4,450 N)

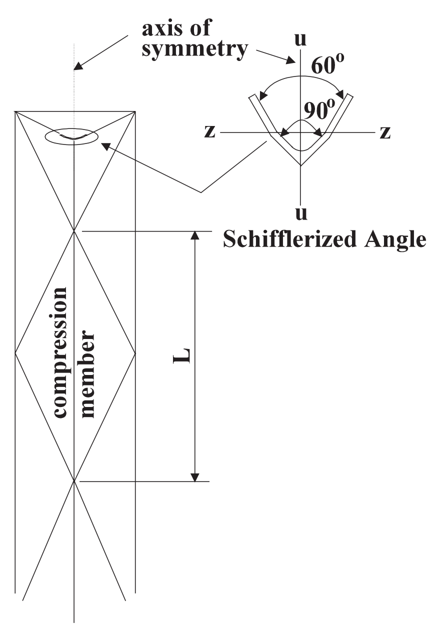


Fig. Ex-15

Schifflerized angles are 90° hot-rolled angles that have had the legs closed to 60°. Properties of Schifflerized angles may be found in *Design Strength of Schifflerized Angle Struts* (Adluri and Madugula 1991).

$L8 \times 8 \times 1/2$ (60°), $L = 9.0$ ft, $F_y = 36$ ksi, $r_u = 2.40$ in., $r_z = 1.83$ in., $u_o = 3.58$ in., $J = 0.646$ in.^{4}, $C_w = 3.68$ in.^{6}, $I_{ps} = 172$ in.^{4}, $A = 7.75$ in.^{2} Note: r_z for the 60° angle is not the commonly tabulated r_z for the hot-rolled shape.

$$\frac{w}{t} = \frac{8-1.125}{0.5} = 13.75; \quad \frac{w}{t_{\lim}} = \frac{80}{\sqrt{36}} = 13.3; \quad \frac{w}{t} > \frac{w}{t_{\lim}}$$

Failure modes include flexural buckling about the u- and z-axis and torsional-flexural buckling about the u-axis. For torsional-flexural buckling, assume that the ends of the member are free to warp at the panel points ($K_t = 1.0$). See Sections 3.7, 3.8.3, 3.9.5, and C3.8.

$$r_t = \sqrt{\frac{3.68 + 0.04(0.646)(1.0 \times 9.0 \times 12)^2}{172}} = 1.33 \text{ in.};$$

$$r_{ps} = \sqrt{\frac{172}{7.75}} = 4.71 \text{ in.}$$

$$\frac{2}{r_{tf}^2} = \frac{1}{(1.33)^2} + \frac{1}{(2.4)^2} + \sqrt{\left(\frac{1}{(1.33)^2} - \frac{1}{(2.4)^2}\right)^2 + 4\left(\frac{3.58}{(1.33)(2.40)(4.71)}\right)^2}; \quad r_{tf} = 1.21$$

$$\text{For } K = 1.0, \quad \frac{KL}{r_u} = 45.0, \quad \frac{KL}{r_z} = 56.0, \quad \frac{KL}{r_{tf}} = 88.8$$

$$F_{cr} = \left[1.677 - 0.677\left(\frac{13.75}{13.3}\right)\right]36 = 35.2 \text{ ksi}$$

$$C_c = \pi\sqrt{\frac{2(29{,}000)}{35.2}} = 127.5$$

$$F_a = \left[1 - 0.5\left(\frac{88.8}{127.5}\right)^2\right]35.2 = 26.7 \text{ ksi}$$

$$\text{Design strength} = 26.7 \text{ ksi} \times 7.75 \text{ in.}^2 = 206.9 \text{ kip.}$$

EXAMPLE 16. SCHIFFLERIZED ANGLE WITH UNSYMMETRICAL BRACING

(1 in. = 25.4 mm; 1 ksi = 6.89 MPa; 1 kip = 4,450 N)

Schifflerized angles are 90° hot-rolled angles that have had the legs closed to 60°. Properties of Schifflerized angles may be found in *Design Strength of Schifflerized Angle Struts* (Adluri and Madugula 1991).

Failure modes include flexural buckling about the u- and z-axis and torsional-flexural buckling about the u-axis. For torsional-flexural buckling, assume that the ends of the member are free to warp at the panel points ($K_t = 1.0$). Since the panel points are staggered, assume that the warping length is based on $L/2$ and solve based on the mixed end condition. Note: r_z for the 60° angle is not the commonly tabulated r_z for the hot-rolled shape.

$L6 \times 6 \times 3/8$ (60°), $L = 9.0$ ft, $F_y = 36$ ksi, $r_u = 1.84$ in., $r_z = 1.44$ in., $u_o = 2.71$ in., $J = 0.204$ in.^{4}, $C_w = 0.698$ in.^{6}, $I_{ps} = 55.7$ in.^{4}, $A = 4.36$ in.^{2}

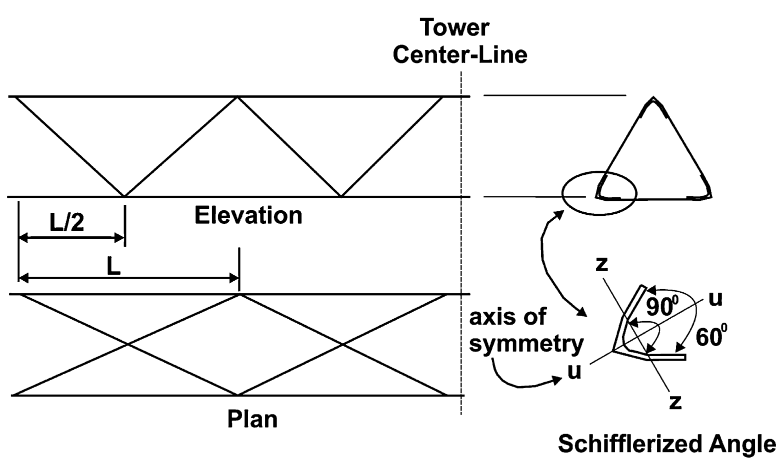
$$\frac{w}{t} = \frac{6-0.875}{0.375} = 13.7; \quad \frac{w}{t_{\lim}} = \frac{80}{\sqrt{36}} = 13.3; \quad \frac{w}{t} > \frac{w}{t_{\lim}}$$

$$r_t = \sqrt{\frac{0.698 + 0.04(0.204)((1.0 \times 0.5) \times 9.0 \times 12)^2}{55.7}} = 0.663 \text{ in.};$$

$$r_{ps} = \sqrt{\frac{55.7}{4.36}} = 3.57 \text{ in.}$$

$$\frac{2}{\left(\frac{r_{tf}}{K'}\right)^2} = \frac{1}{\left(\frac{r_t}{K_t}\right)^2} + \frac{1}{\left(\frac{r_u}{K_u}\right)^2} + \sqrt{\left(\frac{1}{\left(\frac{r_t}{K_t}\right)^2} - \frac{1}{\left(\frac{r_u}{K_u}\right)^2}\right)^2 + 4\left(\frac{u_o}{\left(\frac{r_t}{K_t}\right)\left(\frac{r_u}{K_u}\right)(r_{ps})}\right)^2}$$

Note: r/K values are discussed in Section C3.8.



Lower Chord Member of a Triangular Tower Bridge

Fig. Ex-16

$$\frac{2}{\left(\frac{r_{tf}}{K'}\right)^2} = \frac{1}{\left(\frac{0.663}{(1.0\times0.5)}\right)^2} + \frac{1}{\left(\frac{1.84}{1.0}\right)^2} + \left[\sqrt{\left(\frac{1}{\left(\frac{0.663}{(1.0\times0.5)}\right)^2} - \frac{1}{\left(\frac{1.84}{1.0}\right)^2}\right)^2 + 4\left(\frac{2.71}{\left(\frac{0.663}{(1.0\times0.5)}\right)\left(\frac{1.84}{1.0}\right)(3.57)}\right)^2}\,\right]$$

$$\frac{K'}{r_{tf}} = 0.878$$

$$\frac{L}{r_u} = \frac{9\times12}{1.84} = 58.7; \quad \frac{L}{r_z} = \frac{0.5(9)\times12}{1.44} = 37.5;$$

$$\frac{K'L}{r_{tf}} = 9\times12\times0.878 = 94.8$$

Since the load in the member will be changing at each end of the panel and normal framing eccentricities apply, use Eq. (3.7-7).

$$\frac{KL}{r_{tf}} = 60 + 0.5(94.8) = 107.4$$

$$F_{cr} = \left[1.677 - 0.677\left(\frac{13.7}{13.3}\right)\right]36 = 35.3\text{ ksi}$$

$$C_c = \pi\sqrt{\frac{2(29{,}000)}{35.3}} = 127.3; \quad \frac{KL}{r_{tf}} < C_c$$

$$F_a = \left[1 - 0.5\left(\frac{107.4}{127.3}\right)^2\right]35.3 = 22.7\text{ ksi}$$

$$\text{Design strength} = 22.7\text{ ksi}\times4.36\text{ in.}^2 = 99.0\text{ kip.}$$

EXAMPLE 17. *j*-VALUE DETERMINATION

This example demonstrates the use of Eq. (3.14-4) for determining a *j*-value used for calculating *r* for singly symmetric open sections per Section 3.14.7.

$$\text{Eq. (3.14-4)}, \quad j = \left[\frac{1}{2I_y}\int_A (x^2+y^2)x\,dA\right] - x_o$$

where *x* is the axis of symmetry.

The general solution of the integral $\left[\int_A (x^2+y^2)x\,dA\right]$ gives the following:

$$= \frac{1}{6}\left(y_2^3x_2^2 - y_1^3x_2^2 - y_2^3x_1^2 + y_1^3x_1^2\right) + \frac{1}{4}\left(y_2x_2^4 - y_1x_2^4 - y_2x_1^4 + y_1x_1^4\right)$$

Using the channel shown below,

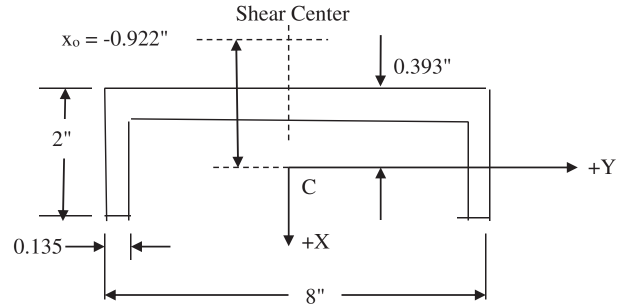


Fig. Ex-17A

Note: The positive direction of the *x*-axis is such that the shear center coordinate x_o is negative.

The channel is divided into three areas as shown below:

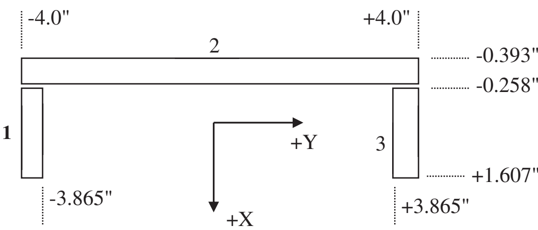


Fig. Ex-17B

Area	Y_1	Y_2	X_1	X_2
1	−4	−3.865	−0.258	+1.607
2	−4	+4	−0.393	−0.258
3	+3.865	+4	−0.258	+1.607

Integral of Area 1:

$$\left[\int_A (x^2+y^2)x\,dA\right]_1 = \frac{1}{6}\Big((-3.865)^3(1.607)^2 - (-4)^3(1.607)^2 - (-3.865)^3(-0.258)^2 + (-4)^3(-0.258)^2\Big) + \frac{1}{4}\Big((-3.865)(1.607)^4 - (-4)(1.607)^4 - (-3.865)(-0.258)^4 + (-4)(-0.258)^4\Big) = 2.5805 + 0.2728 = 2.8533\text{ in.}^5$$

Integral of Area 2:

$$\left[\int_A (x^2+y^2)x\,dA\right]_2 = -1.9137\text{ in.}^5$$

Integral of Area 3:

$$\left[\int_A (x^2+y^2)x\,dA\right]_3 = 2.8533\text{ in.}^5$$

Combined integral = 2.8533 − 1.9137 + 2.8533 = 3.79 in.^{5}

Solving for *j*,

$$j = \left[\frac{1}{2I_y}\int_A (x^2+y^2)x\,dA\right] - x_o = \left[\frac{1}{(2)(0.485)}(3.79)\right] - (-0.922) = 4.83\text{ in.}$$

where

$$I_y = 0.485\text{ in.}^4$$

$$\left[\int_A (x^2+y^2)x\,dA\right] = 3.79\text{ in.}^5$$

$$x_o = -0.922\text{ in.}$$

APPENDIX C
GUIDELINES FOR EXISTING TOWERS

C.1 INTRODUCTION

Many utilities are "upgrading" or "uprating" many of their transmission lines. Modification to the wire system (either conductors or shield wire) requires the utility to assess the structural capability of the transmission towers to sustain the new loads imposed by these changes. The need to meet these new loading criteria has prompted a greater interest in what issues should be considered in analysis of older towers.

Many towers were designed and installed before the current Standards and Guidelines developed by ASCE. Therefore, some of the design requirements within the Standard will not be met by these existing tower designs without extensive changes to the towers. Yet at the same time, many of these towers have provided excellent service life, and these results can be used in a successful upgrade or uprate.

It is recommended that the Standard should be used in development of all tower designs. However, for existing tower designs, the historic performance of the towers should be considered. Every effort should be made to meet the requirements of the Standard when making modifications for new loads on existing towers, but it is not realistic to expect that every facet of the Standard can be applied to older towers. A tower that has historically performed well can provide insight to its ability to support new design loads. In addition, towers that passed full-scale testing can offer additional insight to their capabilities. This information can be valuable during the upgrade or uprate process.

The potential degradation of tower capacity caused by aging, especially in coastal or harsh environments, also needs to be considered when completing a tower upgrade or uprate.

The Standard makes every effort to clearly define a set of rules, and the Commentary provides explanatory material, references, and additional information related to sections of the Standard. The intention of this nonbinding appendix is to provide further insights specific to upgrading or uprating existing towers.

C.2 SLENDERNESS RATIOS

The Standard provides limits for slenderness ratios that may not be met by existing towers. (Section 3.4 provides the specific slenderness limitations.) Therefore, when upgrading older towers, historic performance of the structure with the existing slenderness ratios should be considered. If the member capacity as determined by this Standard, without limitation to the L/r ranges, meets or exceeds the new member force, then it may not be necessary to change or modify the member to meet the slenderness ratio recommendations in the Standard. If recorded performance shows that specific members have been routinely replaced because of fatigue failures, then it is suggested that current slenderness ratio criteria be followed for these members. Any new or replacement members added to the existing tower should meet the requirements of the Standard.

C.3 MINIMUM DISTANCES

Existing towers often meet the minimum end distances, center-to-center bolt hole spacing, and edge distances given in the Standard. If these recommendations are not satisfied, then sound engineering judgment should be used to determine if the original spacing is acceptable under the new loading condition. Testing of a sample of existing members to verify connection capacity is also an alternative. Replacement members should meet the end distances, center-to-center bolt hole spacing, and edge distance criteria recommendations in the Standard.

C.4 BOLT SHEAR CAPACITY

Design documentation for older towers may not include the original assumptions for allowable shear capacity of the bolts. Often, the only information available is the bolt diameter. Therefore, if specific bolt shear capacities are unknown, it is recommended that a random sample of bolts be removed from the existing towers and tested per ASTM F606 (2013). Sample sizes recommended by ASTM A394 (2008) for determination of the mechanical properties vary with the quantity of bolts. It has been suggested that a minimum of 1 bolt for every 10,000 bolts of like diameter and tower manufacturer be tested with a minimum of 10 bolts tested. For long lines with towers fabricated and installed at the same time, an upper limit of 100 bolts seems reasonable. Testing should be completed with threads in the shear plane unless it is determined that details of the existing towers eliminated this possibility (i.e., use of washers or recessed nuts).

C.5 BEARING CAPACITY OF BOLTS AND MEMBERS

Older tower designs often did not address bearing capacity of bolts or members. ASTM A394 Type 0 or Type 1 and A325 bolts (ASTM 2010) are often used in towers. The minimum allowable tensile stress per ASTM is 74 ksi for A394 Type 0 bolts and 120 ksi for A394 Type 1 or A325 bolts up to 1-in. diameter. These tensile stress values exceed the tensile strength of the member (i.e., $F_{u(\min)} = 58$ ksi for A36 steel and $F_{u(\min)} = 65$ ksi for A572-50 steel) (ASTM 2013). Thus at the present time, the allowable bearing stress of new bolt material does not control the bearing capacity. The bearing capacity is controlled by the allowable bearing stress of the member (See Section C4.4 in the Commentary for more discussion on bearing stress). However, bearing capacity may be governed by the bolt bearing capacity on older towers.

C.6 TESTED TOWERS

Section 3.18 states, "Design values other than those prescribed … may be used if substantiated by experimental or analytical investigations." Similarly, Section 3.7.4.6 states. "Where tests and/or analysis demonstrate that specific details provide restraint different from the recommendations … the values of *KL/r* … may be modified." These statements provide some latitude to the designer in meeting the standard criteria when test results can substantiate other values. The original full-scale test data on an older tower can be valuable when making decisions during the tower upgrade or uprate process.

If connection details do not meet current standards, testing of components and connections from existing towers can also be considered. These results may allow the designer an opportunity to use the existing details or modify the detail to meet the load but still not meet the Standard recommendations.

C.7 MEMBER USE RATIOS

During the analysis of older towers, often member loads exceed the calculated design capacity. The overstress in these members is caused in part by using more sophisticated analyses in determining member loads, as well as the differences in calculation of member capacities. In many cases, these members should be reinforced with additional bracing, may require that bolts be added, or in some cases the member should be replaced to meet the current design criteria. However, in some cases it may be acceptable for member use ratios to exceed 100% of capacity for nonlegislated load cases. These cases should be brought to the attention of the owner, including the risk in exceeding the allowable capacities considered. Often, extreme event cases can be reduced slightly to enable member loads to meet the design capacities.

C.8 MAN-LOAD ON HORIZONTAL MEMBERS

OSHA provides recommendations for climbing towers. These criteria may not be met on older towers. The owner should be notified of these limitations, and alternate tower maintenance procedures may be considered.

C.9 MINIMUM SUPPORT OF REDUNDANT MEMBERS

Redundant members in many older towers were normally designed to meet only the *L/r* limits required. The load in the redundant member required to laterally support the braced members was not normally calculated. Upgrading of existing towers often requires additional redundant members to brace the tower leg. These new members should be designed to meet the criteria set forth in Section 3.16 of the Standard. However, it is not reasonable to expect that a tower that has been in service for a number of years should have the complete redundant system replaced to meet this new criterion in Section 3.16 (new compression capacities for redundant members). Again, historic performance should be considered when making changes in redundant members.

C.10 BARS USED AS TENSION HANGERS IN CROSS ARMS

Many older towers used bar stock (solid square, rectangular, or round members as tension-only members). The most frequent application of the bar stock is as hanger members or straps across the tower at the hanger location. In some cases, bars were used as the tension-only members in the X-brace panel. The concern is that these members may have *L/r* ratios that are very large (>>500). Members with high *L/r* ratios have been known to vibrate under low, steady wind conditions. These vibrations over time can cause fatigue cracks at the bolt hole, resulting in failure of the member.

Bar sizes have been used successfully in many older towers. However, if historic performance data indicate that hangers have been replaced because of fatigue failures, it is suggested that replacement members meet the current slenderness ratio criteria. However, if performance has shown no recorded failures, it is likely that the conditions to create fatigue failures do not exist for that particular set of site conditions and design parameters.

C.11 ASTM MATERIAL SPECIFICATIONS USED IN OLDER TOWERS

In many cases, design documentation for older towers may not identify the type of steel used. The following historic information may be useful to the designer. According to the *AISC Manual of Steel Construction: Allowable Stress Design*, 5th Ed. (1947) the specification for A7 steel was a combination of the *Standard Specification for Steel for Bridges* (A7-36) (1936) and for *Standard Specification for Steel for Buildings* (A9-36) (1936). These specifications were originally adopted in 1901 and were combined to become A7 steel in 1937 (1937).

AISC provides seven pages describing the ASTM standard *Specification for Steel for Bridges and Buildings* (ASTM A7-46 1946). For designers, the most important information is the mechanical properties of A7 steel. Tensile strength (F_u) is given as a minimum of 60 ksi to a maximum of 72 ksi. Yield point (F_y) is given as 50% of the tensile strength but in no case less than 33 ksi.

The AISC 6th edition first referenced ASTM A36-63T steel in 1963. Tensile strength (F_u) values were given as 58 ksi to 80 ksi with a yield point (F_y) of 36 ksi. ASTM specifications indicate that A36-60T first was published in 1960. AISC 6th edition also provided the ASTM specification for A7-61T steel in 1961. This was the last edition in which A7 steel was listed. Thus it appears likely that steel used for transmission towers before 1960 was A7 steel, between 1960 and 1970 either A7 or A36, and thereafter a minimum of A36 steel. In the AISC 6th edition, high-strength steels were also introduced. These included ASTM A242-63T (1963), A440-63T (1963), A373-58T (1958), and A441-63T (1963). Therefore, it is possible that one of these high-strength steels was used in existing compression members with low *L/r* values.

In 1964, the Canadian Standards Association (CSA) introduced G40.12 steel. Mechanical properties for G40.12 steel were a minimum tensile strength (F_u) of 62 ksi with a yield point (F_y) of 44 ksi for angles less than 3/4 in. thick. The yield point was decreased to 40 ksi for thicker angle sizes.

Published in 1970, the AISC 7th edition dropped reference to A7-61T (1961) and A373-58T (1958) steels. ASTM A36-70a, A242-70a, A440-70a, and A441-70a (all 1970) were still included as referenced specifications. A572 steel was first referenced by AISC in the 8th edition in 1980 but was originally included in the ASTM specification in 1966. A588 steel was first referenced by AISC in the 7th edition in 1970 but was originally included in ASTM specifications in 1968.

Currently, ASTM A36 (2012) and A572 (2013) are the most prevalent specifications used for steel transmission tower hot-rolled angles and plates in the United States. Canadian suppliers are currently using CSA G40.21 steel in lieu of A36 steel with a minimum tensile strength (F_u) value of 65 ksi and a maximum of 90 ksi and a yield point (F_y) of 44 ksi.

C.12 ORIGINAL COMPRESSION FORMULAS USED IN OLDER TOWER DESIGNS

Before the first edition of the ASCE *Guide for Design of Steel Transmission Towers* (published in 1971), designers often used compression formulas that were inherent to their companies. These formulas were usually straight-line curves resulting in allowable stresses, which many times are lower for values of L/r less than C_c and greater for values of L/r higher than C_c. However, while the allowable stress value (F_a) for the lower values of L/r may be higher using the current standard, K factors should also be included. Older designs normally did not use an adjustment factor (K) to modify the allowable compression stress. Tension values were also calculated without consideration for block shear and at times without checking bearing capacity.

Therefore, older towers often contain members that have a smaller capacity based on the original curves in low L/r ranges. For example, the capacity of leg members using the ASCE standard with $K = 1$ often result in an allowable leg load that is greater than the original design capacity. Using the same L/r with the ASCE standard to determine F_a for a strut that is eccentric at both ends may result in a smaller capacity than determined by the original design formulas because of the addition of the K factor.

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INDEX

Page numbers followed by *e*, *f*, and *t* indicate equations, figures, and tables, respectively.