2018 Interim Revisions to the LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals

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2018 INTERIM REVISIONS INSTRUCTIONS AND INFORMATION

General

AASHTO has issued proposed interim revisions to the *LRFD Structural Supports for Highway Signs, Luminaires, and Traffic Signals*, First Edition (2015). This packet contains the revised pages. They are designed to replace the corresponding pages in the book.

Affected Articles

<u>Underlined text</u> indicates revisions that were approved in 2017 by the AASHTO Highways Subcommittee on Bridges and Structures. ~~Strikethrough text~~ indicates any deletions that were likewise approved by the Subcommittee. A list of affected articles is included below.

All interim pages are displayed on a blue background to make the changes stand out when inserted in the first edition binder. They also have a page header displaying the interim publication year. Please note that these pages may also contain nontechnical (i.e., editorial) changes made by AASHTO publications staff; any changes of this type will not be marked in any way so as not to distract the reader from the technical changes.

2018 Changed Articles

SECTION 3: LOADS
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C3.8.4
C3.8.5

SECTION 4: ANALYSIS AND DESIGN—GENERAL CONSIDERATIONS
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SECTION 10: SERVICEABILITY REQUIREMENTS
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B.2
B.3

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V = basic wind speed, expressed as a 3-s gust wind speed, at 33 ft above the ground in open terrain (mph) (3.2) (3.8.1) (3.8.2) (3.8.7) (C3.8.7)

W_h = wind load on exposed horizontal support (lb) (3.9.2) (3.9.3) (3.9.4.2)

W_l = wind load on luminaires (lb) (3.9.2) (3.9.3) (3.9.4.2)

W_p = wind load on sign panel or traffic signal (lb) (3.9.2) (3.9.3) (3.9.4.2)

W_{sign} = shorter dimension of the attached sign (ft) (3.8.7)

W_v = wind load on exposed vertical supports (lb) (3.9.3)

z = height at which wind pressure is calculated (ft) (3.8.4)

z_g = constant for calculating the exposure factor and is a function of terrain (3.8.4)

α = constant for calculating the exposure factor and is a function of terrain (3.8.4)

3.4—LOAD FACTORS AND LOAD COMBINATIONS

The loads described in Articles 3.5 through 3.8 shall be combined into appropriate load combinations as required in Table 3.4-1. Each part of the structure shall be proportioned for the combination producing the maximum load effect.

The fatigue loads shall be computed in accordance with Section 11.

C3.4

This publication supersedes the AASHTO *Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals* (2009). Beginning with this edition, the design philosophy is based on LRFD. The load and resistance factors are calibrated to provide a reliability index of approximately 3.0 for 300-yr MRI, 3.0 to 3.5 for 700-yr MRI, and 3.5-4.0 for 1700-yr MRI for main members.

These specifications use fatigue limit states I and II for infinite and finite life approaches, respectively. AASHTO LTS Design (Section 11) uses only the infinite life approach for design (fatigue limit state I). The evaluation of the fatigue limit may use a finite life approach to predict the remaining fatigue life for asset management purposes.

NCHRP Report 796 outlines the calibration. (Puckett, et al., 2014)

Table 3.4-1—Load Combinations and Load Factors

Load Combination Limit State	Description	Reference Articles	Permanent: Dead Components (DC)		Transient: Live Load (LL)	Transient: Wind (W)	Fatigue: Natural Wind Gust Vibration (NWG)	Fatigue: Vortex-Induced Vibration (VVW)	Fatigue: Combined Wind on High-Level Towers	Fatigue: Galloping Induced Vibration (GVW)
			Max/Min	Mean			Apply Separately			
Strength I	Gravity	3.5, 3.6, and 3.7	1.25		1.6					
Extreme I	Wind	3.5, 3.8, 3.9	1.1/0.9			1.0[a]				
Service I	Translation	10.4		1.0		1.0[b]				
Service II	Serviceability Check	10.4.2.1		1.0						
Service II	Crack control for Prestressed Concrete			1.0		1.00				
Fatigue I	Infinite-life	11.7		~~1.0~~			1.0	1.0	1.0	1.0
Fatigue II	Evaluation	17.5		~~1.0~~			1.0	1.0	1.0	1.0

a Use Figures 3.8-1, 3.8-2, or 3.8-3 (for appropriate return period)

b Use Figure 3.8-4 (service)

3.5—PERMANENT LOADS

The permanent load shall consist of the weight of the structural support, signs, luminaires, traffic signals, lowering devices, and any other appurtenances permanently attached to and supported by the structure. Temporary loads during inspection and maintenance shall also be considered as part of the permanent loads.

C3.5

In the these specifications, the terms permanent load or dead load may be used interchangably. Dead load is to include all permanently attached fixtures, including hoisting devices and walkways provided for servicing of luminaires or signs.

The points of application of the weights of the individual items may be their respective centers of gravity.

Manufacturers' data may be used for the weights of components.

3.6—LIVE LOADS

A live load consisting of a single load of 0.5 kips distributed over 2.0 ft transversely to the member shall be used for designing members for walkways and service platforms.

C3.6

The specified live load represents the weight of a person and equipment during servicing of the structure. Only the members of walkways and service platforms are designed for the live load. Any structural member designed for the combined loadings in Article 3.4 will be adequately proportioned for live load application. For OSHA-compliant agencies, additional requirements may apply.

Typically, live load will not control the design of the structural support.

3.7—ICE LOAD–ATMOSPHERIC ICING

Atmospheric ice load due to freezing rain or in-cloud icing may be applied around the surfaces of the structural supports, traffic signals, horizontal supports, and luminaires; but it may be considered only on one face of sign panels.

The Owner shall specify any special icing requirements that occur, including those in and near mountainous terrain, gorges, the Great Lakes, and Alaska.

C3.7

NCHRP Report 796 illustrates that ice and wind on ice does not practically control the critical load effect. To simplify these Specifications, these load combinations have been eliminated. (Puckett et al. 2014)

For extreme cases where the Owner indicates, either local conditions or the ice and coincident wind loads provided ASCE/SEI 7 may be used for guidance. (e.g. ASCE/SEI 7, 2010).

3.8—WIND LOAD

Wind load shall be based on the pressure of the wind acting horizontally on the supports, signs, luminaires, traffic signals, and other attachments computed in accordance with Articles 3.8.1 through 3.8.7. Eq. 3.8.1-1 using the appropriate mean recurrence interval basic wind speed as shown in Figures 3.8-1, 3.8-2, 3.8-3, and 3.8-4. The mean recurrence interval is determined with Table 3.8-1.

C3.8

The selection of the MRI accounts for the consequences of failure. A "typical" support could cross the travelway during a failure thereby creating a hazard for travelers (MRI = 700 yrs). The Owner should specify the ADT and Risk Category (or MRI).

All supports that could cross lifeline travelways are assigned a high risk category to consider the consquences of failure (MRI = 1700 yrs).

Supports that cannot cross the travelway are assigned a low risk and 300-yr MRI.

3.8.3—Special Wind Regions

The wind speed maps presented in Figures 3.8-1, 3.8-2, and 3.8-3 show several special wind regions. If the site is located in a special wind region, or if special local conditions exist in mountainous terrain and gorges, the selection of the basic wind speed should consider localized effects. Where records or experience indicate that wind speeds are higher than those reflected in Figures 3.8-1, 3.8-2, or 3.8-3, the basic wind speed should be increased using information approved by the Owner.

C3.8.3

If the wind speed is to be determined through the use of local meteorological data, ASCE/SEI 7 presents procedures for analyzing local meteorological data. Such increases in wind speed should be based on judgment and the analysis of regional meteorological data. In no case shall the basic wind speed be reduced below that presented in Figures 3.8-1, 3.8-2, or 3.8-3.

3.8.4—Height and Exposure Factor K_z

The height and exposure factor K_z shall be determined either from Table C3.8.4-1 or calculated using Eq. 3.8.4-1:

$$K_z = 2.0\left(\frac{z}{z_g}\right)^{\frac{2}{\alpha}} \tag{3.8.4-1}$$

where z is the height above the ground at which the pressure is calculated or 16 ft, whichever is greater, and z_g and α are constants that vary with the exposure condition. Based on ASCE/SEI 7, α should be taken to be 9.5 and z_g should be taken to be 900 ft for Exposure C.

C3.8.4

K_z is a height and exposure factor that varies with height above the ground depending on the local exposure conditions and may be conservatively set to 1.0 for heights less than 33 ft. The variation is caused by the frictional drag offered by various types of terrain.

ASCE/SEI 7-10 defines acceptable wind design procedures using different terrain exposure conditions. For a specified set of conditions, the wind pressures associated with the different exposures increase as the exposure conditions progress from B to D, with exposure B resulting in the least pressure and exposure D resulting in the greatest pressure. Exposure C has been adopted for use in these Specifications as it represents open terrain with scattered obstructions. Exposure coefficients for other terrain conditions may be used per ASCE/SEI 7-10 with permission of the Owner.

Once the terrain exposure conditions are established, the height and exposure factor, K_z, is calculated using the relationship that is presented in ASCE/SEI 7-10.

Table C3.8.4-1 presents the variation of the height and exposure factor, K_z, as a function of height. The coefficient 2.01 in ASCE/SEI 7-10 is rounded to 2.0 in Eq. 3.8.4-1.

Table C3.8.4-1—Height and Exposure Factors, K_z[a]

Height z, ft	K_z
~~≥15~~ <15	0.84
20	0.90
30	0.98
40	1.04
50	1.09
60	1.13
70	1.17
80	1.20
90	1.23
100	1.26
110	1.28
120	1.31
130	1.33
140	1.35
150	1.37

[a] See Eq. 3.8.4-1 for calculation of K_z. (Exposure C)

3.8.5—Directionality Factor K_d

The directionality factor is defined in Table 3.8.5-1.

Table 3.8.5-1—Directionality Factors, K_d

Support Type	Directionality Factor
High-mast and Pole	
Round	0.95
Square	0.90
Octagonal	0.95
Dodecagonal	0.95
Hexdecagonal	0.95
Traffic Signal	0.85
Dynamic Message Sign	0.85
Overhead Frame/Truss	0.85
Support with horizontal arms or members supporting sign and/or signals	0.85

C3.8.5

The directionality factor accounts for ~~two effects: the maximum wind can come from any direction and the probability that the maximum drag coefficient is associated with the wind direction is reduced~~ the reduced probability that the design event wind direction aligns with the most aerodynamically vulnerable direction of the structure. ~~These~~ The directionality factor values are consistent with those from ASCE 7-10 as based upon work by Ellingwood 1981 and Ellingwood et al. 1982.

In previous versions of these specifications, the directionality factor was not considered and thus implicitly set as 1.0 for all supports. Because the directionality factor values in Table 3.8.5-1 result in less conservative designs than those for previous specifications, the Owner may elect to use K_d = 1.0 in lieu of the table values.

4.8.1—Simplified Method

In the combined resistance equations for steel and aluminum (in Article 5.12 and Article 6.12), the bending load effect shall be multiplied by the coefficient for amplification, B_2, to account for the secondary moment. The coefficient for amplification, B_2, may be taken as:

$$B_2 = \frac{1}{1-\left[\dfrac{P_{equivalent}}{P_{Euler\ bottom}}\right]} \geq 1.0 \qquad (4.8.1\text{-}1)$$

where

$$P_{equivalent} = \sqrt[3]{\frac{I_B}{I_t}}\,P_t + 0.38D_P$$

$$P_{Euler\ bottom} = \frac{\pi^2 EI_B}{(kL)^2}$$

where:

P_t = factored vertical concentrated load at the top of the pole (kip),

D_P = factored weight of the pole (kip),

I_B = moment of inertia for the cross-section at the base of the pole (in.^{4}),

I_T = moment of inertia for the cross-section at the top of the pole (in.^{4}),

k = slenderness factor,

L = length of the pole (in.), and

r = radius of gyration (in.).

Eq. 4.8.1-1 is valid where:

$$\cancel{2\pi\sqrt{\frac{E}{F_y}} \leq \frac{kL}{r}} \quad 1.414\pi\sqrt{\frac{E}{F_y}} \leq \frac{kL}{r}$$

If this limit is exceeded, analysis by a detailed method is required.

C4.8.1

The coefficient for amplification B_2 is included in the Specifications to be used mainly for vertical cantilever supports over 55 ft in height or where other conditions are such that secondary P-Δ effects are significant.

The term B_2 is traditionally used to represent moment magnification due to second-order load effects due to P-Δ. For example see, AISC, ACI, and/or *AASHTO LRFD Bridge Design Specifications*.

The term B_1 is traditionally used to represent moment magnification based upon axial softening and chord-slope deformation, i.e., non-sway. This type of magnification is small in the types of supports addressed in these specifications and can be ignored under the limiting slenderness required in this article.

Eq. 4.8.1-1 is limited to values of kL/r greater than or equal to:

$$\cancel{2\pi\sqrt{\frac{E}{F_y}}} \quad 1.414\pi\sqrt{\frac{E}{F_y}}$$

to ensure that the maximum axial stress is limited to $0.25F_y$, where F_y is the specified minimum yield strength and E is the modulus of elasticity. This requirement is intended to keep the axial stresses sufficiently low such that effects of residual stresses on the buckling behavior of the pole can be ignored. The radius of gyration, r, may be calculated at a distance of $0.50L$ for a tapered column. The slenderness factor, k, may be assigned 2.0 for this approximation.

4.8.2—Detailed Method

In lieu of the approximate procedure of Article 4.8.1, a second-order elastic analysis that is applicable to all materials covered by the Specifications may be performed considering the final deflected position of the vertical support. Limit state load factors shall be applied. All loads may be assumed to be applied simultaneously.

C4.8.2

As an alternative procedure, a more rigorous method of analysis is presented in NCHRP Report 796 (Puckett et al., 2014), whereby the member is analyzed considering the actual deflected shape. With this method, the coefficient of amplification, B_2, is taken as 1.0 because the secondary moment is included in the load effects directly from analysis. This method implies a nonlinear relationship between the applied loads and the resulting deflections.

4.9—REFERENCES

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ACI. 2014. *Building Code Requirements for Structural Concrete*. ACI 318-14. American Concrete Institute, Farmington Hills, MI.

AISC. 2010. Manual of Steel Construction, 14th Ed. American Institute of Steel Construction, Chicago, IL.

Barker and Puckett. 2013. *Design of Highway Bridges–An LRFD Approach*, third edition. Wiley, New York, NY.

Fouad, F. H., E. A. Calvert, and E. Nunez. 1998. *Structural Supports for Highway Signs, Luminaires, and Traffic Signals*, NCHRP Report 411. Transportation Research Board, National Research Council, Washington, DC.

Puckett, J., M. Garlich, M. Barker, A. Nowak, C. Menzemer, 2014. *Development and Calibration of AASHTO-LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals*, NCHRP Report 796. Transportation Research Board, National Research Council, Washington, DC.

SECTION 5: STEEL DESIGN

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SECTION 5

STEEL DESIGN

5.1—SCOPE

This Section specifies design provisions for steel structural supports. Fatigue-sensitive steel support structures are further addressed in Section 11. Additional design provisions not addressed in this Section shall be obtained from other references as noted.

Design provisions are provided for round and multi-sided tubular shapes, I-shaped sections, channels, plates, angles, and anchor bolts above the foundation. Anchorage requirements are specified in Section 15.

C5.1

Laminated structures may be used when the fabrication process is such that adequate shear transfer between the lamina can be achieved. Their use is subject to the approval of the Owner.

5.2—DEFINITIONS

Anchor Bolt—A bolt, stud, or threaded rod used to transmit loads from the attachment into the concrete support or foundation. The end cast in concrete shall be provided with a positive anchorage device, such as forged head, nut, hooked end, or attachment to an anchor plate to resist forces on the anchor bolt.

Anchorage—The process of attaching a structural member or support to the concrete structure by means of an embedment, taking into consideration those factors that determine the load capacity of the anchorage system.

Attachment—The structural support external to the surfaces of the embedment that transmits loads to the embedment.

Compact Section—A section capable of developing the plastic moment capacity.

Ductile Anchor Connection—A connection whose resistance is controlled by the strength of the steel anchorage rather than the strength of the concrete.

Ductile Anchor Failure—A ductile failure occurs when the anchor bolts are sufficiently embedded so that failure occurs by yielding of the steel anchor bolts.

Embedment—The portion of a steel component embedded in the concrete used to transmit applied loads from the attachment to the concrete support or foundation.

Headed Anchor—A headed bolt, a headed stud, or a threaded rod with an end nut.

High-Mast Lighting Tower (HMLT)—Pole-type tower that provides lighting at heights greater than 55 ft.

Lateral-Torsional Buckling (LTB)—The buckling mode of a flexural member involving deflection normal to the plane of bending that occurs simultaneously with twist about the shear center of the cross section.

Local Flange Buckling (LFB)—Section instability due to buckling of flange or other local part of the cross section.

Multi-sided Tube—A section with generally round characteristics having eight or more sides.

Noncompact Section—A section in which the moment capacity is not permitted to exceed its yield moment.

Rectangular Tube—A square or rectangular section (four sides). Resistance checks differ from multi-sided tubes.

Retrofit Anchor Bolt—An anchor that is installed into hardened concrete.

Slender Section—A section in which the moment capacity is governed by buckling prior to reaching its yield moment.

5.3—NOTATION

A_e = effective net area (in.^{2}) (5.9.2) (5.9.3)

A_{EFF} = effective area summation (in.^{2}) (5.10.2.3)

A_g = gross area (in.^{2}) (5.9.2) (5.9.3) (5.10.2.3) (5.11.2.1.1) (5.11.2.1.2) (5.12.1)

A_n = net area (in.^{2}) (5.9.3)

A_v = shear area (in.^{2}) (5.11.2) (5.11.2.1.1) (5.11.2.1.2) (5.11.2.2)

A_w = area of the web (in.^{2}) (5.11.2.2)

a_w = ratio of two times the web area in compression due to application of major axis bending moment alone to the area of the compression flange components (5.8.3.2.4)

B = ratio (5.8.4.4) (5.12.1)

B = moment magnification factor (5.12.1)

B_x = moment magnification factor for second order effects (x axis) (5.12.1) (5.12.2)

B_y = moment magnification factor for second order effects (y axis) (5.12.1) (5.12.2)

b = element width (in.) (5.7.2) (C5.7.2) (5.7.3) (5.8.2) (5.8.3.1.2) (5.8.3.1.3) (5.10.2.3) (5.11.2.2)

b_e = element effective width (in.) (5.10.2.3)

b_f = flange width of rolled beam (in.) (5.7.3) (C5.9.3)

b_l = longer leg width (in.) (5.10.2.4)

b_s = shorter leg width (in.) (5.10.2.4)

C_b = moment gradient coefficient (5.8.3.1.3) (5.8.3.2.4) (5.8.7.2)

C_J = the torsional constant (5.11.3) (C5.11.3)

C_v = shear buckling coefficient (5.11.2.2)

C_w = warping constant (in.^{6}) (5.8.3.1.3)

c = lateral-torsional buckling section coefficient (5.8.3.1.3)

D = inside diameter of round cross-section (in.) (5.6.3) (5.6.6.1) (5.7.2) (C5.7.2) (5.8.2) (5.10.2.2) (5.11.2.1.1) (5.11.3.1.1)

D = outside diameter of round cross section (in.) (5.6.2) (C5.6.2) (5.6.3) (5.6.6.1) (5.7.2) (C5.7.2) (5.8.2)

d = full nominal depth for stems of tees (in.) (5.7.3) (5.8.4.3) (5.8.4.4) (5.8.7.1) (5.8.7.2) (C5.9.3) (5.11.2.2)

d = full nominal depth for webs of rolled or formed sections (in.) (5.7.3) (5.8.4.3) (5.8.4.4) (5.8.7.1) (5.8.7.2) (C5.9.3)

E = modulus of elasticity of steel, 29,000 (ksi) (5.7.2) (5.7.3) (5.8.2) (5.8.3.1.3) (5.8.3.2.4) (5.8.4.3) (5.8.4.4) (5.8.7.1) (5.8.7.2) (5.10.2.1) (5.10.2.2) (5.10.2.3) (5.11.2.1.1) (5.11.2.2) (5.11.3.1.1) (5.12.1)

F_{cr} = critical buckling stress (ksi) (5.8.3.1.1) (5.8.3.2.4) (5.8.4.3) (5.8.7.2) (5.10.2.1) (5.10.2.3)

F_e = Euler stress, calculated in the plane of bending (ksi) (5.10.2.1)

F_L = stress defined in Table 5.7.3-1 (ksi) (5.7.3)

F_{nt} = torsional resistance (ksi) (5.11.3) (5.11.3.1.1) (5.11.3.1.2) (5.11.3.2)

F_{nv} = nominal shear resistance (ksi) (5.11.2) (5.11.2.1.1) (5.11.2.1.2) (5.11.2.2) (5.11.3.2)

F_u = specified minimum fracture stress (ksi) (5.9.2)

F_y = specified minimum yield stress (ksi) (5.7.2) (5.7.3) (5.8.2) (5.8.3.1.1) (5.8.3.1.2) (5.8.3.1.3) (C5.8.3.1.3) (5.8.3.2.1) (5.8.3.2.2) (5.8.3.2.3) (5.8.3.2.4) (5.8.4.2) (5.8.5.1) (5.8.5.2) (5.8.7.1) (5.8.7.2) (5.9.2) (5.10.2.1) (5.10.2.2) (5.11.2.1.1) (5.11.2.1.2) (5.11.2.2) (5.11.3.1.2)

f = buckling stress (ksi) (5.10.2.3)

G = elastic shear modulus (ksi) (5.8.4.4)

g = transverse center-to-center spacing (gauge) between lines of fasteners (in.) (5.9.3)

H = height of backing ring at a groove-welded tube-to-transverse-plate connection (in.) (5.6.5) (C5.6.5)

h = clear distance between the flanges for webs of rolled or built-up sections (in.) (5.7.3) (5.11.2.2)

h_c = twice the distance from center of gravity to inside face of compression flange (in.) (5.7.3) (5.8.3.2.4)

h_o = distance between flange centroids (in.) (5.8.3.1.3) (5.8.3.2.4)

h_p = twice the distance from plastic neutral axis to inside face of compression flange (in.) (5.7.3)

I = moment of inertia of the bolt group (in.^{4}) (5.8.4.4)

I_y = out-of-plane moment of inertia (in.^{4}) (5.8.3.1.3) (5.8.3.2.1) (5.8.3.2.3) (5.8.4.4)

I_{yc} = out-of-plane moment of inertia of the compression flange (in.^{4}) (5.8.3.2.1) (5.8.3.2.3)

J = torsional constant (in.^{4}) (5.8.3.1.3) (5.8.3.2.4) (5.8.4.4)

K = effective length factor (5.10.2.1) (C5.10.2.1) (5.10.2.4) (5.10.3) (5.12.1)

k_c = slenderness ratio (5.7.3)

k_s = buckling factor (5.11.2.2)

L = distance between cross-sections braced against twist or lateral displacement of the compression flange. For cantilevers braced against twist only at the support, L may conservatively be taken as the actual length (in.). (5.9.4) (C5.10.2.1) (5.10.2.4) (5.10.3) (5.12.1)

L = length of column (in.) (5.9.3) (5.9.4) (C5.10.2.1) (5.10.3) (5.10.2.4) (5.11.3.1.1) (5.12.1)

L = length of connection in the direction of loading (in.) (5.9.3) (5.9.4) (5.10.2.1) (C5.10.2.1) (5.10.2.4) (5.10.3) (5.11.3.1.1) (5.12.1)

L_b = unbraced length of member (in.) (5.8.3.1.3) (5.8.3.2.4) (5.8.4.4.) (5.8.7.1) (5.8.7.2)

L_p = lateral-torsional buckling length limit (in.) (5.8.3.1.3) (C5.8.3.1.3) (5.8.3.2.4)

L_r = lateral-torsional buckling length limit (in.) (5.8.3.1.3) (5.8.3.2.4)

L_v = distance from maximum to zero shear force (in.) (5.11.2.1.1)

L_w = length of weld (in.) (5.9.3)

M_A = absolute value of the quarter-point moment in the unbraced segment (kip-in.) (5.8.3.1.3)

M_B = absolute value of the center-point moment in the unbraced segment (kip-in.) (5.8.3.1.3)

M_C = absolute value of the third quarter-point moment in the unbraced segment (kip-in.) (5.8.3.1.3)

M_{cr} = lateral-torsional buckling moment (5.8.4.4)

M_{max} = absolute value of the maximum moment in the unbraced segment (kip-in.) (5.8.3.1.3)

M_n = nominal flexural resistance (kip-in.) (5.8.1) (5.8.2) (5.8.3.1.1) (5.8.3.1.2) (5.8.3.1.3) (5.8.3.2.2) (5.8.3.2.3) (5.8.3.2.4) (5.8.4.1) (5.8.4.2) (5.8.4.3) (5.8.4.4) (5.8.5.1) (5.8.5.2) (5.8.7.1) (5.8.7.2)

M_{nx} = nominal flexural resistance about x axis (kip-in.) (5.12.2)

M_{ny} = nominal flexural resistance about y axis (kip-in.) (5.12.2)

M_p = plastic flexural resistance (kip-in.) (5.7.3) (5.8.2) (5.8.3.1.1) (5.8.3.1.2) (5.8.3.2.1) (5.8.3.2.3) (5.8.4.1) (5.8.4.2) (5.8.5.1) (5.8.5.2) (5.8.7.1) (5.8.7.2)

M_{px} = plastic flexural resistance about x axis (kip-in.) (5.12.2)

M_{py} = plastic flexural resistance about y axis (kip-in.) (5.12.2)

M_r = factored flexural resistance (kip-in.) (5.8.1) (5.8.3.2.4) (5.12.1)

M_{rx} = factored flexural resistance about x axis (kip-in.) (5.12.1) (5.12.2)

M_{ry} = factored flexural resistance about y axis (kip-in.) (5.12.1) (5.12.2)

M_u = factored bending moment (kip-in.) (5.12.1)

M_{ux} = factored bending moment about x axis (kip-in.) (5.12.1) (5.12.2)

M_{ux}^* = factored bending moment skewed diagonal loading about x axis (kip-in.) (5.12.1) (5.12.2)

M_{uy} = factored bending moment about y axis (kip-in.) (5.12.1) (5.12.2)

M_{uy}^* = factored bending moment from skewed diagonal loading about y axis (kip-in.) (5.12.1) (5.12.2)

M_y = yield moment (kip-in.) (5.7.3) (5.8.4.1) (5.8.4.2) (5.8.7.1) (5.8.7.2)

M_{yc} = yield moment based upon compression yielding (kip-in.) (5.8.3.2.1) (5.8.3.2.2) (5.8.3.2.4)

M_{yt} = yield moment based upon tension yielding (kip-in.) (5.8.3.2.3)

n = number of sides on a multi-sided section (5.6.2) (5.7.2) (C5.7.2)

P_e = Euler elastic buckling capacity (kip) (5.12.1)
P_{nc} = minimum nominal compressive strength (kip) (5.10.1) (5.10.2.1)
P_{nl} = minimum nominal compressive strength (kip) (5.12.1)
P_{nu} = minimum nominal rupture strength on the effective net area (kip) (5.9.1) (5.9.2)
P_{ny} = minimum nominal tensile strength from limit states of yield on gross sectional area (kip) (5.9.1) (5.9.2)
P_r = factored nominal axial resistance (kip) (5.12.1)
P_{rc} = minimum nominal compressive strength (kip) (5.10.1)
P_{rt} = minimum nominal tension strength (kip) (5.9.1)
P_u = factored axial load (kip) (5.12.1)
Q = local buckling adjustment factor (5.10.2.1) (C5.10.2.1) (5.10.2.2) (5.10.2.3)
R = root gap at a groove-welded tube-to-transverse-plate connection (in.) (5.6.5)
R_n = general nominal resistance (5.13)
R_{nr} = factored nominal resistance of cables (kip) (5.13)
R_{pc} = moment ratio for compression flange yielding (5.8.3.2.1) (5.8.3.2.2) (5.8.3.2.4)
R_{pt} = moment ratio for tension flange yielding (5.8.3.2.3)
R_{rt} = factored tensile resistance of cables (kip) (5.13)
r = governing radius of gyration (in.) (5.9.4) (5.10.2.1) (C5.10.2.1) (5.10.2.4) (5.10.3) (5.12.1) (5.13)
r_b = inside bend radius of a plate (in.) (C5.7.2)
r_t = radius of gyration of a section comprising the compression flange plus $^1/_3$ of the compression web area, taken about an axis in the plane of the web (in.) (5.8.3.2.4)
r_{ts} = radius of gyration on effective section (in.) (5.8.3.1.3)
r_x = radius of gyration about the axis parallel to the connected leg (in.) (5.10.2.4)
r_y = radius of gyration about the y-axis (in.) (5.8.3.1.3) (C5.8.3.1.3)
r_z = radius of gyration about the minor principle axis (in.) (5.10.2.4)
S_x = elastic section modulus (in.^{3}) (5.8.3.1.2) (5.8.3.1.3) (C5.8.3.1.3) (5.8.3.2.1) (5.8.3.2.4) (5.8.4.3) (5.8.7.2) (C5.12.2)
S_{xc} = elastic section modulus for outer compression fibers (in.^{3}) (5.7.3) (5.8.3.2.1) (5.8.3.2.2) (5.8.3.2.3) (5.8.3.2.4)
S_{xt} = elastic section modulus for outer tension fibers (in.^{3}) (5.7.3) (5.8.3.2.1) (5.8.3.2.2) (5.8.3.2.3)
S_y = elastic section modulus (in.^{3}) (5.8.5.1) (5.8.5.2)
s = longitudinal center-to-center spacing (pitch) of any two consecutive holes (in.)
T_n = nominal torsional resistance (kip-in.) (5.11.1) (5.11.3)
T_r = factored torsional resistance (kip-in.) (5.11.1)
T_u = factored torque (kip-in.) (5.12.1)
t = wall thickness or thickness of element (in.) (5.6.5) (C5.6.5) (5.6.6.1) (5.7.2) (C5.7.2) (5.7.3) (5.8.2) (5.8.3.1.2) (5.8.7.1) (5.8.7.2) (5.10.2.2) (5.10.2.3) (5.11.2.1.1) (5.11.2.2) (5.11.3.1.1)
t_w = thickness of web (in.) (5.7.3) (5.8.4.3) (5.11.2.2)
U = shear lag coefficient (5.9.3) (C5.9.3)
V_n = nominal shear resistance (kip) (5.11.1) (5.11.2)
V_r = factored shear resistance (kip) (5.11.1) (5.12.1)
V_{rx} = factored shear resistance parallel to the x axis (kip) (5.12.1)
V_{ry} = factored shear resistance parallel to the y axis (kip) (5.12.1)
V_u = factored shear (kips) (5.12.1)
V_{ux} = factored shear parallel to the x axis (5.12.1)
V_{uy} = factored shear parallel to the y axis (5.12.1)
W = clear width of reinforced and unreinforced holes and cutouts (in.) (5.6.6.1)
w = width of plate (distance between welds) (in.) (5.9.3)
$\bar{x}$ = connection eccentricity (in.) (5.9.3)

$\bar{x}$ = connection eccentricity (in.) (5.9.3)

Z = plastic section modulus (5.8.7.1)

Z_x = plastic section modulus about the x axis (in.^{3}) (5.8.2) (5.8.3.1.1) (5.8.3.1.3) (C5.8.3.1.3) (5.8.3.2.1) (5.8.4.3) (C5.12.2)

Z_y = plastic section modulus about the y axis (in.^{3}) (5.8.5.1)

λ = width–thickness ratio of the element (5.8.2) (5.8.3.1.2) (C5.8.3.2) (5.8.3.2.1) (5.8.3.2.2) (5.8.3.2.3) (5.8.4.2) (5.8.4.3) (5.8.5.2) (5.10.2.1) (5.10.2.2) (5.10.2.3) (5.12.2)

λ_{max} = maximum width–thickness ratio (C5.7.1) (5.7.2) (5.7.3) (C5.11.3.1.2) (5.12.2)

λ_p = width–thickness ratio at the compact limit (5.7.1) (C5.7.1) (5.7.2) (5.7.3) (5.8.2) (5.8.3.1.2) (5.8.3.2.1) (5.8.3.2.2) (5.8.3.2.3) (5.8.4.2) (5.8.4.3) (5.8.5.2)

λ_r = width–thickness ratio at the noncompact limit (5.7.1) (C5.7.1) (5.7.2) (5.7.3) (5.8.2) (5.8.3.1.2) (C5.8.3.2) (5.8.3.2.1) (5.8.3.2.3) (5.8.5.2) (5.10.2.1) (5.10.2.2) (5.10.2.3) (5.12.2)

θ = angle of the sound beam for ultrasonic inspection of groove welds (degrees) (5.5.3.2) (5.6.5)

ϕ_c = resistance factor for compression (5.5.3.2) (5.10.1) (5.12.1)

ϕ_f = resistance factor for flexure (5.5.3.2) (5.8.1) (5.12.2)

ϕ_n = resistance factor for cables (5.13)

ϕ_T = resistance factor for torsion (5.5.3.2)

ϕ_t = resistance factor for bolt tension (5.11.1) (5.12.1)

ϕ_u = resistance factor for tension fracture (5.5.3.2) (5.9.1)

ϕ_v = resistance factor for shear (5.5.3.2) (5.11.1)

ϕ_y = resistance factor for tension yield (5.5.3.2) (5.9.1)

5.4—MATERIAL

Grades of steel listed in the *AASHTO LRFD Bridge Design Specifications* (LRFD Design) are applicable for welded structural supports for highway signs, luminaires, and traffic signals.

For steels not generally addressed by LRFD Design, but having a specified yield strength acceptable to the Owner, the LRFD limit state design criteria shall be derived by applying the general equations given in LRFD Design except as indicated by this Section.

All steels greater than 0.5 in. in thickness, used for structural supports for highway signs, luminaires, and traffic signals, that are main load carrying tension members shall meet the current Charpy V-Notch impact requirements in LRFD Design.

C5.4

Steel other than that listed may be used with permission from the Owner.

Typical steel materials used in structural supports for highway signs, luminaires, and traffic signals are:

- ASTM A595 Grade A, B, and C
- ASTM A572 Grade 42, 50, 55, 60, and 65
- ASTM A1011
- ASTM F1554 Grade 36, 55, and 105 Anchor Bolts

Generally, the Specification indicated in this Section applies.

Although the structural supports addressed by these Specifications are not subjected to high-impact loadings, steel members greater than 0.5 in. in thickness should meet a general notch toughness requirement to avoid brittle fracture. The non-fracture critical values may be used.

5.5—DESIGN LIMIT STATES

5.5.1—General

Structural components and connections shall be proportioned to satisfy the requirements at strength, extreme event, service, and fatigue limit states.

5.5.2—Service Limit State

General service requirements are provided in Section 10.

5.5.3—Strength Limit State

5.5.3.1—General

Strength and stability shall be considered using the applicable strength load combinations specified in Table 3.4-1.

C5.5.3.1

NCHRP project 10-80 developed specific LRFD load and resistance factors using ASCE/SEI 07-2010 loading.

5.5.3.2—Resistance Factors

Resistance factors, ϕ, for the strength limit states shall be taken as follows:

- Flexure $\phi_f = 0.90$
- Shear $\phi_v = 0.90$
- Torsion $\phi_T = 0.95$
- Axial compression, $\phi_c = 0.90$
- Tension, fracture in net section $\phi_u = 0.75$
- Tension, yielding in gross section $\phi_y = 0.90$

C5.5.3.2

NCHRP Project Report 796 determined resistance factors specifically for signs, luminaires, and traffic signal supports and these may differ from other specifications. (Puckett et al., 2014)

5.5.4—Extreme Limit State

All applicable load combinations in Table 3.4-1 for the extreme event limit state shall be investigated.

The resistance factors for the extreme event shall be as defined in the strength limit state in Article 5.5.3.

C5.5.4

The ASCE 7-10 wind maps were generated using a wind load factor of unity. These specifications use a similar approach and therefore include wind in combination with other loads to be addressed as an extreme event.

5.5.5—Fatigue Limit State

Components and details shall be investigated for fatigue as specified in Section 11.

5.6—GENERAL DIMENSIONS AND DETAILS

5.6.1—Minimum Thickness of Materials

The minimum thickness of material for main supporting members of steel truss-type supports shall be 0.1793 in. For secondary members, such as bracing and truss webs, the minimum thickness shall be 0.125 in. The minimum thickness of material for all members of pole-type supports and truss-type luminaire

C5.6.1

Main members are those that are strictly necessary to ensure integrity of a structural system. Secondary members are those that are provided for redundancy of the system and stability of components and members. Minimum thickness requirements may be based on service considerations such as corrosion resistance as arms shall be 0.125 in. These limits may be reduced no more than ten percent for material designated by gauge numbers.

Steel supports for small roadside signs may be less than 0.125 in. in thickness.

specified by the Owner. The thickness 0.1793 in. is associated with 7 gauge sheet steel material.

Supports without an external breakaway mechanism that have thicknesses less than 0.125 in. have shown good safety characteristics in that they readily fail under vehicle impact, with little damage to the vehicle or injury to the occupants. These thinner supports should be used on those installations considered to have a relatively short life expectancy such as small roadside signs.

5.6.2—Minimum Number of Sides

Tubular structures shall be of round or multi-sided cross-section. Cross-sections with concave external surface, such as "fluted" cross-sections, are not covered by the provisions for multi-sided cross-sections and shall not be used without approval of the Owner. Multi-sided tubular sections shall have a minimum number of sides as stated in the following equation, and a minimum internal bend radius of five times the tube wall thickness or 1 in., whichever is larger.

$$n \geq \sqrt{5D} \qquad (5.6.2\text{-}1)$$

where:

D = outside distance from flat side to flat side of multi-sided tubes (in.), and

n = greater than or equal to the square root of $5D$ or 8, whichever is larger.

C5.6.2

Fatigue cracking in multi-sided tube-to-transverse-plate connections initiates at the bend corners and progresses towards the flat face between the corners. Research has demonstrated the existence of high stress concentration at the bend corners, which caused crack initiation early during the fatigue tests in eight-sided tubes with sharp bend radii (Roy et al., 2011).

Compared to a round tube of similar size, welded connections in multi-sided tubes with fewer sides and internal bend radii less than 1 in. exhibited significantly less fatigue resistance. Increasing the number of sides and/or increasing the internal bend radius can improve fatigue performance of multi-sided sections (Roy et al., 2011).

The requirement of minimum number of sides for multi-sided tubes was derived considering a maximum $^1/_2$ in. radial distance between the multi-sided tube and its inscribed circle at a corner. The $^1/_2$ in. distance was chosen based on the performance of specimens that were fatigue tested in the laboratory. For flat-to-flat distance D of multi-sided cross-sections, the minimum number of sides is conservatively provided as:

D up to 13 in.

8 sides (octagonal)

D greater than 13 in. and up to 28 in.

12 sides (dodecagonal)

D greater than 28 in. and up to 50 in.

16 sides (hexadecagonal)

Although there is no clear consensus among the research community, multi-sided galvanized tubes employing very sharp bend radii can be susceptible to strainage embrittlement, cold-worked embrittlement, and hydrogen embrittlement leading to early fatigue cracking in service. A minimum bend radius of five times the tube wall thickness can mitigate the possibility of such embrittlement.

Square or rectangular sections are susceptible to early fatigue cracking leading to poor fatigue performance. These sections should not be used for highway sign, signal, and high-level luminaire support structures. (Dexter and Ricker, 2002)

Fluted cross sections and other complex hollow sections present a variety of issues that may affect the design and performance of structures. Owner approval of poles or mast arms using fluted and other complex hollow sections should require verification of the materials, design methods, detailing requirements, fabrication methods, and fatigue performance by analysis and testing of components and systems.

5.6.3—Transverse Plate Thickness

The base plate thickness shall be considered in the design of tube-to-transverse-plate connections. In addition, for arms or pole bases of supports that are designed according to Section 11, the minimum plate thickness shall be as provided in Table 5.6.3-1.

Table 5.6.3-1—Minimum Transverse Plate Thickness for Fatigue

Section Diameter or depth D, in.	Minimum Plate Thickness, in.
$D \leq 8$	1.5
$D > 8$	2.0

C5.6.3

Experimental and analytical studies (Koenigs et al., 2003; Hall, 2005; Warpinski, 2006; Ocel et al., 2006; Roy et al., 2011; Stam et al., 2011) demonstrated that the fatigue resistance of tube-to-transverse-plate connections is a function of the relative flexibility of the tube and the transverse plate. Transverse plate flexibility has a major impact on stress amplification in the tube wall adjacent to the weld toe. Increasing the transverse plate thickness is the most cost-effective means of reducing the flexibility of the transverse plate and increasing the connection fatigue resistance. In-service fatigue cracking in tube-to-transverse-plate connections often occurred where relatively thin plates were used along with a few discrete fasteners.

Reducing the opening size in the transverse plate and/or increasing the number of fasteners are other cost-effective means of reducing the flexibility of the transverse plate and increasing the fatigue resistance of tube-to-transverse-plate connections. In laboratory tests (Roy et al., 2011), groove-welded tube-to-transverse-plate connections exhibited significantly better fatigue resistance compared to fillet-welded connections in identical structures, because a smaller opening could be used in the transverse plate.

Fluted cross sections and other complex hollow sections present a variety of issues that may affect the design and performance of structures. Owner approval of poles or mast arms using fluted and other complex hollow sections should require verification of the materials, design methods, detailing requirements, fabrication methods, and fatigue performance by analysis and testing of components and systems.

5.6.4—Stiffened Base Connection

In stiffened fillet-welded tube-to-transverse-plate connections (socket connections) only tapered stiffeners having a termination angle of 15 degrees on the tube shall be used.

The minimum height of stiffeners shall be 12 in. At least eight stiffeners shall be used equally spaced around the tube wall. The stiffener spacing shall not exceed 16 in.

When stiffened fillet-welded tube-to-transverse-plate connections are used, the minimum thickness of the tube wall shall be 0.25 in.

The ratio of the stiffener thickness to the tube-wall thicknesses shall not exceed 1.25.

C5.6.4

In support structures employing larger diameter and thicker tubes, optimized stiffened tube-to-transverse-plate fillet-welded connections can provide a cost-effective design compared to an increased transverse plate thickness. Parametric studies (Roy et al., 2011) demonstrated that the fatigue performance of a stiffened connection is a function of the geometric parameters of the connection: the tube thickness, the transverse plate thickness, the stiffener shape and size (thickness, height, and angle), and the number of stiffeners (or stiffener spacing). A large stiffener thickness relative to the tube can attract more stress into the stiffeners and can increase distortion of the tube. By contrast, relatively

Stiffeners having a transition radius shall not be used.

Alternative stiffener geometries, stiffener spacing, and weld termination angles on the tube shall be approved by the Owner based upon evaluation, analysis, testing, or acceptable field performance, singly or in combination.

thin stiffeners can reduce distortion of the tube but fail to sufficiently reduce the stress at the fillet-weld and can cause fatigue cracking through the throat of the stiffener-to-transverse plate weld.

A ratio of stiffener thickness to tube thickness of 1.25 provides an optimum solution with equal likelihood of fatigue cracking at the stiffener termination and at the tube-to-transverse-plate weld.

Decreasing the ratio of the stiffener height to stiffener spacing reduces protection to the fillet-weld. An optimum solution is obtained when the stiffener height is about 1.6 times the stiffener spacing.

Reducing the termination angle of the stiffener on the tube wall improves the fatigue performance of stiffened connections. Using a stiffener termination angle of 15 degrees ensures that the stiffener sections are fully effective in sharing load.

Stiffeners with a transition radius at the termination on the tube wall are fabrication intensive and are expected to be costlier than a tapered alternative. To avoid exposure of the lack of fusion at the weld root in fillet welds and partial-penetration groove welds, a stiffener termination with a transition radius must be groove welded, which requires non-destructive inspection in the vicinity of weld termination. It is difficult to grind the weld toe without inadvertently thinning the tube at the transition.

The stiffened groove-welded tube-to-transverse-plate connection is unlikely to be cost-effective and is excluded from this specification.

Figures of stiffeners are illustrated in Table 11.9.3.1-1. (Detail 6.2 and 6.3)

5.6.5—Backing Ring

For full-penetration groove-welded tube-to-transverse-plate connections, the thickness of the backing ring shall not exceed $^1/_4$ in. The height of the backing ring, when welded to the tube at the top prior to performing ultrasonic inspection of the groove weld, shall be as given by Eq. 5.6.5-1, rounded to the nearest integer:

$$H = 2t(\tan\theta) + R \qquad (5.6.5\text{-}1)$$

where:

H = height of backing ring at a groove-welded tube-to-transverse-plate connection (in.),

R = root gap at a groove-welded tube-to-transverse-plate connection (in.),

θ = angle of the sound beam for ultrasonic inspection of groove welds (degrees), and

t = the tube wall thickness (in.).

C5.6.5

In full-penetration groove-welded tube-to-transverse-plate connections with the backing ring welded to the plate and the tube wall, fatigue cracking can occur both at the groove-weld toe and the backing ring top weld toe on the tube wall. Depending on the diameter and thickness of the tube, and the height and thickness of the backing ring, the backing ring can participate in transferring forces from the tube to the transverse plate and can introduce variability in the fatigue performance of the connection. Providing a 2 in. × $^1/_4$ in. backing ring limits this participation to a reasonable level in typical support structures.

However, when the backing ring is welded to the tube at the top, this weld interferes with the ultrasonic inspection of the groove weld by allowing the sound wave to travel from the outside of the shaft through the weld into the backing ring. The sound wave then gets trapped in the backing ring and does not reach the groove weld. For a successful inspection, the weld at the top of the backing ring should be above the centerline of the probe. According to AWS D1.1, the ultrasonic beam should bounce at least once to the area of inspection.

For tube-to-transverse-plate connection employing an external collar, the tube thickness for the above equation shall include the thickness of the collar and the tube.

When the top weld of the backing ring is made after the ultrasonic inspection of the groove weld, or when the backing ring is not welded at the top, the height of the backing ring shall not exceed 2 in.

which creates a full "V" signal. From experience, a shallow beam angle such as 70 degrees produces the best results. Thus, for thicker tubes with a 45-degree bevel and a root gap, the probe placement gets higher and therefore the backing ring needs to be taller.

The backing ring heights for different tube thicknesses are tabulated in the following table for a root gap of $^1/_4$ in. and an angle of ultrasonic beam of 70°.

Table C5.6.5-1—Required Backing Ring Height

t, in.	H, in.
$t \leq 0.3125$	2
$0.3125 < t \leq 0.50$	3
$0.50 < t \leq 0.6875$	4

This requirement for backing ring height is not applicable if the top weld of the backing ring is made after the ultrasonic inspection of the groove weld or if the backing ring is not welded at the top. In such cases, a maximum 2-in.-high backing ring will be sufficient. Also refer to Section 14 for additional recommendations regarding welding the backing ring to the tube.

When welded to the tube, the backing ring provides a redundant load path when the tube-to-transverse-plate groove weld develops fatigue cracking.

5.6.6—Holes and Cutouts

5.6.6.1—Unreinforced and Reinforced Holes and Cutouts

Unreinforced and reinforced holes and cutouts shall be detailed as shown in Figure 5.6.6.1-1, Figure 5.6.6.1-2, and Figure 5.6.6.1-3. The width of opening in the cross sectional plane of the tube shall not be greater than 40 percent of the tube diameter D at that section for structures that are designed according to Section 11. The corners of the opening shall be rounded to radius as shown. In the figures below, double-headed arrows indicate termination is beyond the view illustrated.

C5.6.6.1

In laboratory fatigue tests (Roy et al., 2011), fatigue cracking from unreinforced hand holes in sign/signal support structure specimens initiated from the edge of hand hole at the point of maximum stress concentration. The hand holes in the test specimens were located in the plane of the mast-arm but on the away face to produce the most critical stress condition in the hand hole detail for fatigue.

It is recommended that in sign/signal support structures the hand holes and other holes and cutouts be located in a region of low stress. Since the fatigue stress cycles in sign/signal support structures are imparted primarily due to wind-induced galloping oscillations in the plane containing the arm, it is recommended that the hand holes be located on the side at 90 degrees to that containing the cantilever arm (Figure C5.6.6.1-1). The hand hole may be located on either side.

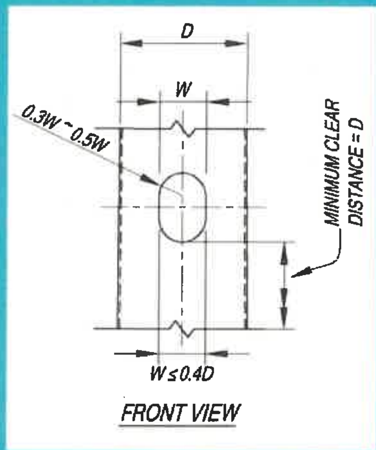


Figure 5.6.6.1-1—Details of Unreinforced Holes and Cutouts

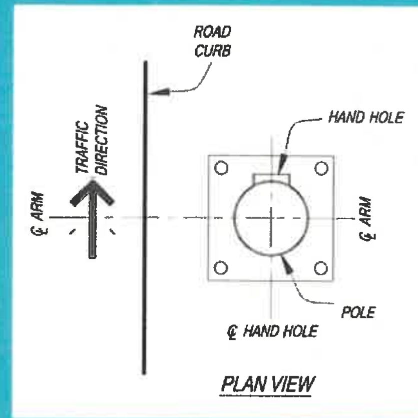


Figure C5.6.6.1-1—Recommended Orientation of a Hand Hole

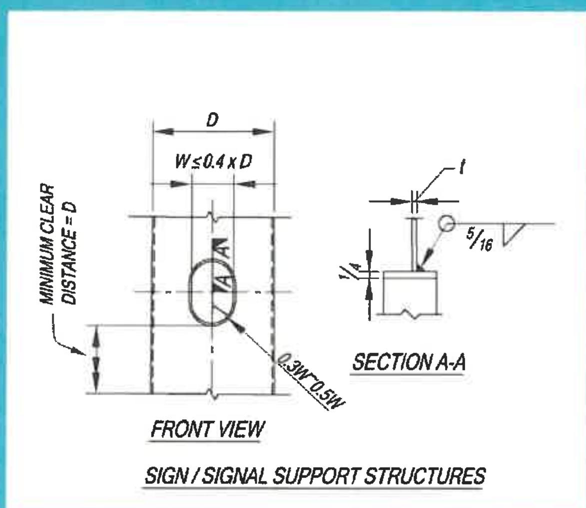


Figure 5.6.6.1-2—Details of Reinforced Holes and Cutouts

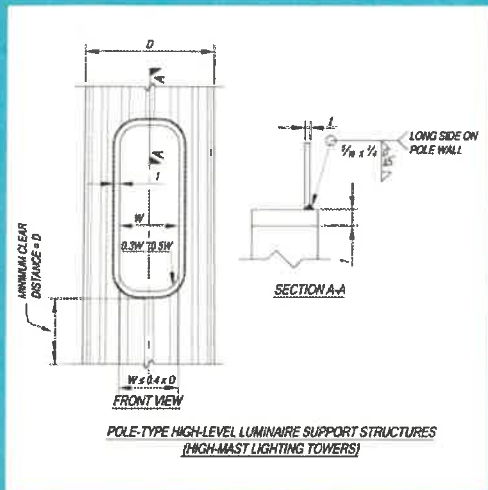


Figure 5.6.6.1-3—Details of Reinforced Holes and Cutouts for High-Mast Poles

Alternative geometries shall be approved by the Owner based upon evaluation, analysis, testing, or acceptable field performance, singly or in combination. Location of cutouts and appurtenances shall be approved by the Owner based on sound engineering practices.

In service, fatigue cracks at reinforced hand holes have been reported from the toe of the hand hole frame-to-pole (reinforcement-to-tube) weld in high level lighting support structures. In laboratory fatigue tests (Roy et al., 2011), fatigue cracking from hand hole details in sign/signal support structure specimens initiated only from the lack of fusion at the root of the hand hole frame-to-pole (reinforcement-to-tube) fillet-weld. Because of limited access, the hand hole frames in sign and signal structures can be welded only from the outside, increasing the possibility of lack of fusion defects at the weld root.

The hand holes in the test specimens were located in the plane of the mast-arm but on the away face such as to produce the most critical stress condition in the hand hole detail for fatigue. Since the fatigue stress cycles in sign/signal support structures are imparted primarily due to wind-induced galloping oscillations in the plane containing the arm, it is recommended that the hand holes be located on the side at 90° to that containing the arm. In high level lighting support structure specimens, the hand hole details did not develop any fatigue cracking (Roy et al., 2011).

5.6.7—Mast-Arm-to-Pole Connections

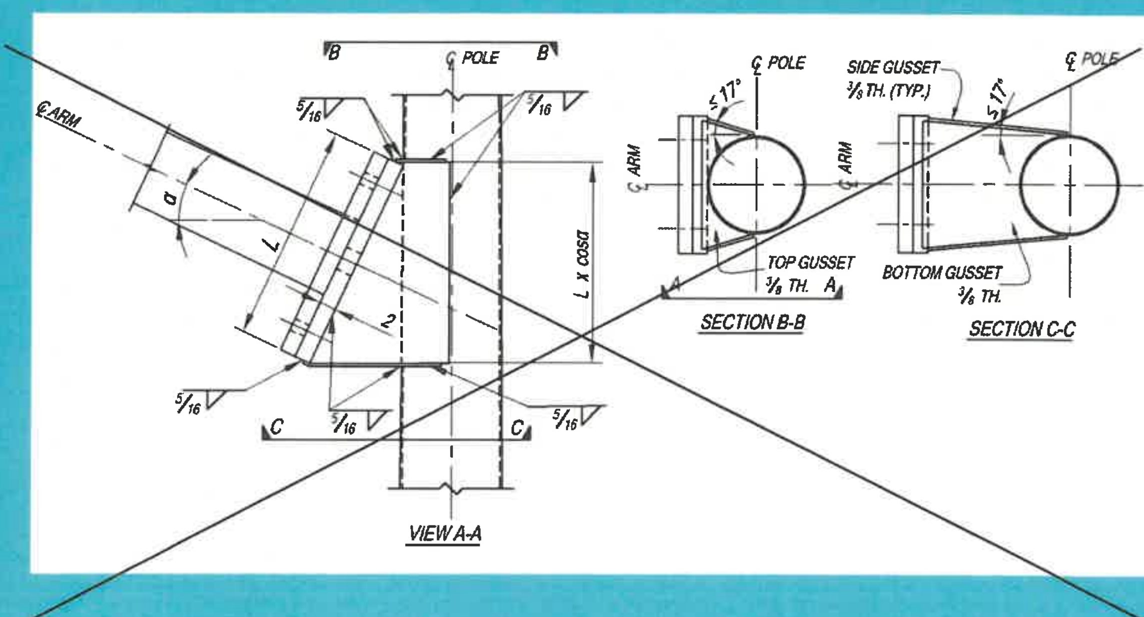
Mast-arm-to-pole connections employing fillet-welded gusseted box or ring-stiffened box have been shown to be effective and fatigue resistant. Connections validated with testing ~~may be used and~~ are encouraged. Fillet-welded gusseted box connections shall be limited to a D/t ratio of 50.

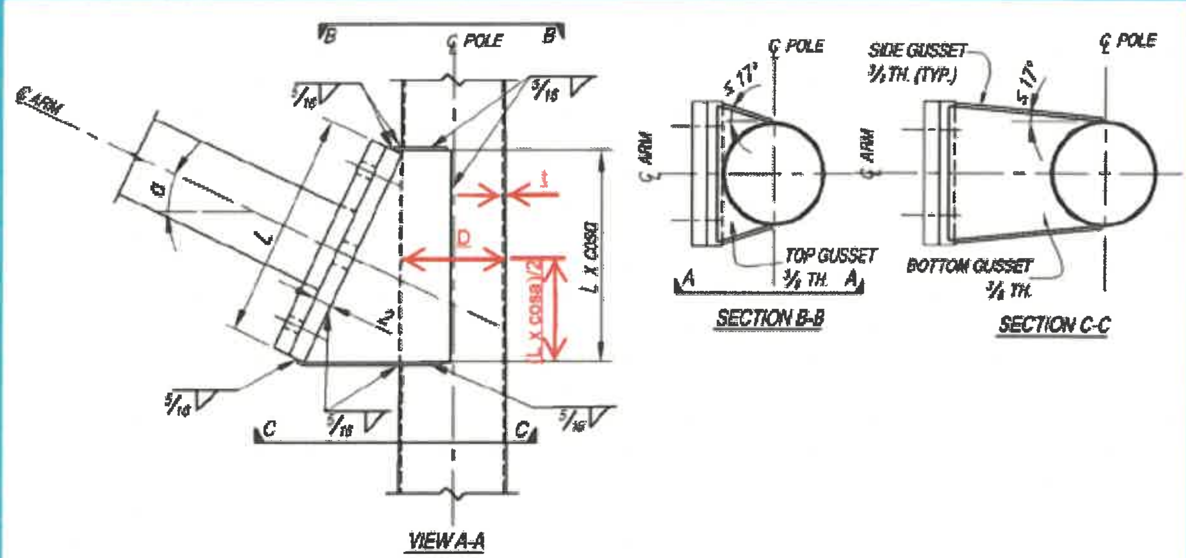
C5.6.7

Fillet-welded gusseted boxes or ring-stiffened boxes at the mast-arm-to-pole connections tested in the laboratory in full size specimens (Roy et al., 2011) did not develop any fatigue cracking under both in-plane and out-of-plane loading. These connections were tested at various load levels and in some specimens were subjected to in excess of 40 million stress cycles. In all specimens, fatigue cracking occurred in other critical details in the structure, such as the tube-to-transverse-plate welds in the mast-arm and/or the pole, and/or hand holes.

In-service fatigue cracking of ~~these connections~~ fillet-welded gusseted box connections on larger diameter poles has been reported. Fatigue testing has shown the advantage of ring stiffeners that completely encircle a pole relative to a built-up box connection. Fillet-welded gusseted box connections performed well in fatigue testing where D/t was limited to 50 or less (Roy et al., 2011). For tapered tubes, D shall be measured at mid-height of the connection. For built-up box connections, it is recommended that the width of the box be at least the same as the diameter of the column (i.e., the sides of the box are tangent to the sides of the column).

~~Ring-stiffened box connections are more fabrication intensive and should be employed in geographic regions where support structures are expected to experience significant wind induced oscillations. In other regions, gusseted box connections are expected to provide satisfactory performance.~~ See Figures C5.6.7-1 and C5.6.7-2 for details.





Fillet-welded gusseted box connections shall be limited to a D/t ratio of 50.

Figure C5.6.7-1—Details of Fillet-Welded Gusseted Box Connections

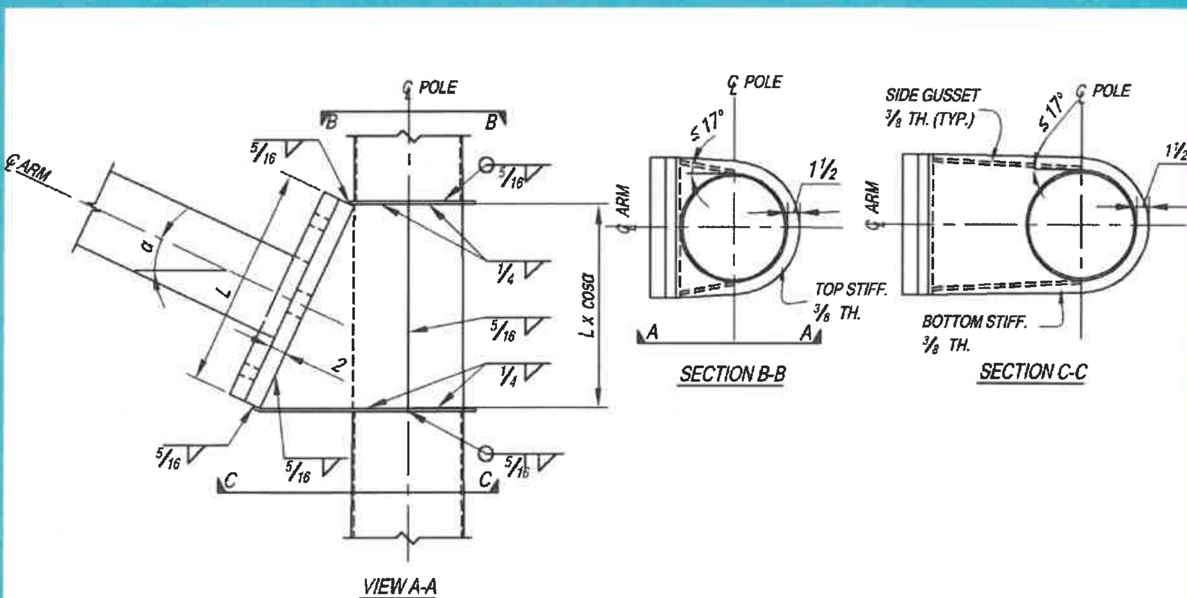


Figure C5.6.7-2—Details of Fillet-Welded Ring-Stiffened Box Connections

5.6.8—Slip Type Field Splice

The minimum length of any telescopic (i.e., slip type) field splices for all structures shall be 1.5 times the inside diameter of the exposed end of the female section.

C5.6.8

ASCE/SEI 48-11 (2011) for the design transmission poles provides a more rigorous approach that may be consulted for guidance.

5.7—SECTION CLASSIFICATION FOR LOCAL BUCKLING

5.7.1—Classification of Steel Sections

Steel sections are classified as compact, noncompact, and slender element sections. For a section to qualify as compact or noncompact, the width–thickness ratios of compression elements must not exceed the applicable corresponding limiting λ_p or λ_r values given in Tables 5.7.2-1 and 5.7.3-1, respectively. If the width–thickness ratios of any compression element section exceed the noncompact limiting value, λ_r, the section is classified as a slender element section.

C5.7.1

Cross section elements with width–thickness ratios greater than the limits in Tables 5.7.2-1 and 5.7.3-1 may experience local buckling. Flexural members may be subject to local buckling when the width–thickness ratio exceeds λ_p. Members in compression may be subject to local buckling when the width–thickness ratio exceeds λ_r.

5.7.2—Width–Thickness Ratios for Round and Multi-sided Tubular Sections

The limiting diameter–thickness, D/t, ratios for round sections and width–thickness, b/t, ratios for multi-sided tubular sections are given in Table 5.7.2-1.

For multi-sided tubular sections, the effective width, b, is the inside distance between intersection points of the flat sides less:

$$\tan\left(\frac{180}{n}\right) \qquad (5.7.2\text{-}1)$$

times the minimum of the inside bend radius or $4t$, on each side. If the bend radius is not known, the element width, b, may be calculated as the inside width between intersection points of the flat sides less:

$$(3t)\tan\left(\frac{180}{n}\right) \qquad (5.7.2\text{-}2)$$

C5.7.2

The equation for the element width of multi-sided tubular sections may be calculated as:

$$b = \tan\left(\frac{180}{n}\right)\left[D' - 2t - \text{minimum}\left(2r_b, 8t\right)\right] \qquad (C5.7.2\text{-}1)$$

where:

D' = the outside distance from flat side to flat side of multi-sided tubes and $180/n$ is in degrees.

Table 5.7.2-1—Width–Thickness Ratios for Round and Multi-Sided Tubular Sections

Shape	Ratio	λ_p	λ_r	λ_{max}
Round	D/t	$0.07\frac{E}{F_y}$	$0.11\frac{E}{F_y}$ *(compression)* $0.31\frac{E}{F_y}$ *(flexure)*	$0.45\frac{E}{F_y}$
Hexadecagonal (16)	b/t	$1.12\sqrt{\frac{E}{F_y}}$	$1.26\sqrt{\frac{E}{F_y}}$	$2.14\sqrt{\frac{E}{F_y}}$
Dodecagonal (12)	b/t	$1.12\sqrt{\frac{E}{F_y}}$	$1.41\sqrt{\frac{E}{F_y}}$	$2.14\sqrt{\frac{E}{F_y}}$
Octagonal (8)	b/t	$1.12\sqrt{\frac{E}{F_y}}$	$1.53\sqrt{\frac{E}{F_y}}$	$2.14\sqrt{\frac{E}{F_y}}$
Flanges of Square/Rectangle	b/t	$1.12\sqrt{\frac{E}{F_y}}$	$1.53\sqrt{\frac{E}{F_y}}$	$2.14\sqrt{\frac{E}{F_y}}$

5.7.3—Width–Thickness Ratios for Compression Plate Elements

Limiting width–thickness ratios for nontubular shapes are given in Table 5.7.3-1.

Plate elements are considered unstiffened or stiffened, depending on whether the element is supported along one or two edges, parallel to the direction of the compression force, respectively.

For unstiffened elements, which are supported along one edge parallel to the direction of the compression force, the width shall be taken as:

- b = half the full nominal flange width, b_f, for flanges of I-shaped members and tees (in.),
- b = the full nominal dimension for legs of angles and flanges of channels (in.), and
- d = the full nominal depth for stems of tees (in.).

For stiffened elements, which are supported along two edges parallel to the direction of the compression force, the width shall be taken as:

- h = the clear distance between flanges for webs of rolled or built-up sections (in.),
- d = the full nominal depth for webs of rolled or built-up sections (in.),
- h_c = twice the distance from center of gravity to inside face of compression flange (in.), and
- h_p = twice the distance from plastic neutral axis to inside face of compression flange (in.).

The unsupported width of such elements shall be taken as the distance between the nearest lines of fasteners or welds, or between the roots of the flanges in the case of rolled sections, or as otherwise specified in this Article.

C5.7.3

For cross sections element not listed in Table 5.7.3-1, the AISC *Manual of Steel Construction* (2011) should be used to determine the limiting width–thickness ratios. The maximum width–thickness ratio would be the applicable λ_r limit.

Table 5.7.3-1—Width–Thickness Ratios for Nontubular Sections

Description	Width/Thickness	λ_p	$\lambda_r = \lambda_{max}$
Flexure of flanges of rolled I-shapes, tees, and channels	b/t	$0.38\sqrt{\frac{E}{F_y}}$	$1.0\sqrt{\frac{E}{F_y}}$
Uniform compression in I-shape flanges and channel flanges	b/t	N/A	$0.56\sqrt{\frac{E}{F_y}}$
Uniform compression of single angles and double angles and all other unstiffened elements	b/t	N/A	$0.45\sqrt{\frac{E}{F_y}}$
Uniform compression in stems of tees	d/t	N/A	$0.75\sqrt{\frac{E}{F_y}}$
Uniform compression of webs of I-shapes and all other stiffened elements	h/t_w, b/t	N/A	$1.49\sqrt{\frac{E}{F_y}}$
Flexure in legs of angles	b/t	$0.54\sqrt{\frac{E}{F_y}}$	$0.91\sqrt{\frac{E}{F_y}}$
Flexure in stems of tees	d/t	$0.84\sqrt{\frac{E}{F_y}}$	$1.03\sqrt{\frac{E}{F_y}}$
Flexure in webs of doubly symmetrical I-shapes	h/t_w	$3.76\sqrt{\frac{E}{F_y}}$	$5.70\sqrt{\frac{E}{F_y}}$
Flexure in webs of singly symmetrical I-shapes and built-up I-shapes	h_c/t_w	$\dfrac{\frac{h_c}{h_p}\sqrt{\frac{E}{F_y}}}{\left(0.54\frac{M_p}{M_y}-0.09\right)} \le \lambda_r$	$5.70\sqrt{\frac{E}{F_y}}$
Flexure in flanges of singly symmetrical I-shapes and built-up I-shapes	b/t	$0.38\sqrt{\frac{E}{F_y}}$	$0.95\sqrt{\frac{k_c E}{F_L}}$

Notes:

h_c = twice the distance from center of gravity to inside face of compression flange

h_p = twice the distance from plastic neutral axis to inside face of compression flange

$k_c = 4/\sqrt{h/t_w} \le 0.76$

$F_L = 0.7F_y$ when $S_{xt}/S_{xc} \ge 0.7$; $F_L = F_y\, S_{xt}/S_{xc} \ge 0.5$ when $S_{xt}/S_{xc} \le 0.7$

5.7.4—Slender Element Sections

Except as allowed for round and multi-sided tubular sections, compression plate elements that exceed the noncompact limit specified in Table 5.7.3-1 shall not be permitted.

5.8—COMPONENTS IN FLEXURE

5.8.1—General

The provisions of this article apply to flexure of rolled open, tubular, and built-up plate sections.

The factored flexural resistance, M_r, shall be:

$$M_r = \phi_f M_n \tag{5.8.1-1}$$

where:

M_n = nominal flexural resistance, and

ϕ_f = resistance factor as specified in Article 5.5.3.2.

The nominal resistance is determined by Articles 5.8.2 through 5.8.7 as applicable.

C5.8.1

NCHRP Report 796 used previous research to develop LRFD nominal strength equations for typical sign, luminaire, and signal supports. Past allowable stress design equations were modified to limit state design equations for tubular members. AISC (2011) design equations have been incorporated for typical structures. For structures not addressed in these Specifications, other resources should be considered such as AISC (2011) and LRFD Design.

5.8.2—Nominal Bending Strength for Round and Multi-Sided Tubular Members

For round and multi-sided tubular members that have compact, noncompact, and slender element sections as defined in Table 5.7.2-1, the nominal bending strength, M_n, shall be computed according to Table 5.8.2-1.

The nominal bending strength for multi-sided tubes shall not exceed the nominal bending strength for round tubes of equivalent diameter. The equivalent diameter for a multi-sided tube shall be the outside distance between parallel sides.

Table 5.8.2-1—Nominal Bending Strength, M_n, for Tubular Members

Shape	Compact	NonCompact	Slender
	$\lambda \le \lambda_p$	$\lambda_p < \lambda \le \lambda_r$	$\lambda > \lambda_r$
Round	$M_n = M_p = Z_x F_y$	$M_n = M_p\left[0.77 + \dfrac{0.016(E/F_y)}{D/t}\right]$	$M_n = M_p\left[\dfrac{0.25(E/F_y)}{D/t}\right]$
Hexadecagonal	$M_n = M_p = Z_x F_y$	$M_n = M_p\left[2.59 - \dfrac{1.43(b/t)}{\sqrt{E/F_y}}\right]$	$M_n = M_p\left[1.12 - \dfrac{0.26(b/t)}{\sqrt{E/F_y}}\right]$
Dodocagonal	$M_n = M_p = Z_x F_y$	$M_n = M_p\left[1.77 - \dfrac{0.69(b/t)}{\sqrt{E/F_y}}\right]$	$M_n = M_p\left[1.15 - \dfrac{0.25(b/t)}{\sqrt{E/F_y}}\right]$
Octagonal	$M_n = M_p = Z_x F_y$	$M_n = M_p\left[1.50 - \dfrac{0.45(b/t)}{\sqrt{E/F_y}}\right]$	$M_n = M_p\left[1.14 - \dfrac{0.22(b/t)}{\sqrt{E/F_y}}\right]$
Square and Rectangular	$M_n = M_p = Z_x F_y$	$M_n = M_p\left[1.37 - \dfrac{0.33(b/t)}{\sqrt{E/F_y}}\right]$	$M_n = M_p\left[1.23 - \dfrac{0.23(b/t)}{\sqrt{E/F_y}}\right]$

5.8.3—Nominal Bending Strength for Flanged I-Shaped Members, Channels, and Rectangular Tubes

This article applies to singly or doubly symmetric beams loaded in the plane of symmetry. It also applies to channels loaded in a plane passing through the shear center parallel to the web or restrained against twisting at load points and points of support.

C5.8.3

Rectangular tubes are torsionally stiff; however, in an application where the member unbraced length is long, lateral-torsional buckling should be checked.

5.8.3.1—Doubly Symmetric I-Shape, Channel, and Rectangular Tubes Members Bent about Strong Axis with Compact Webs

The nominal bending strength shall be the lower value obtained according to the limit states of plastic moment, lateral-torsional buckling (LTB), and flange local buckling (FLB).

5.8.3.1.1—Plastic Moment

$$M_n = M_p = Z_x F_y \qquad (5.8.3.1.1\text{-}1)$$

where:

Z_x = plastic section modulus (in.^{3}), and

F_y = yield stress (ksi).

C5.8.3.1.1

Web buckling is not included because d/t ratios do not exceed λ_r. Elastic element buckling limits ($\lambda > \lambda_r$) do not need to be considered, except for rectangular tubes, because slender elements are not allowed.

5.8.3.1.2—Flange Local Buckling

If $\lambda \le \lambda_p$ FLB does not apply.

If $\lambda > \lambda_p$

$$M_n = M_p - \left(M_p - 0.7F_yS_x\right)\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right) \qquad (5.8.3.1.2\text{-}1)$$

where:

S_x = section modulus, (in.^{3}),

F_y = yield stress, (ksi), and

λ_p and λ_r are defined in Table 5.7.3-1.

For noncompact or slender rectangular tubes where $\lambda > \lambda_p$, M_n is determined from Table 5.8.2-1 using b/t of the flange.

5.8.3.1.3—Lateral-Torsional Buckling

If $L_b \leq L_p$

$$M_n = M_p = Z_xF_y \qquad (5.8.3.1.3\text{-}1)$$

If $L_p \leq L_b \leq L_r$

$$M_n = C_b\left[M_p - \left(M_p - 0.7F_yS_x\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \leq M_p \qquad (5.8.3.1.3\text{-}2)$$

If $L_b > L_r$

$$M_n = F_{cr}S_x \leq M_p \qquad (5.8.3.1.3\text{-}3)$$

$$F_{cr} = \frac{C_b\pi^2E}{\left(\frac{L_b}{r_{ts}}\right)^2}\sqrt{1 + 0.078\frac{Jc}{S_xh_o}\left(\frac{L_b}{r_{ts}}\right)^2} \qquad (5.8.3.1.3\text{-}4)$$

$$L_p = 1.76r_y\sqrt{\frac{E}{F_y}} \qquad (5.8.3.1.3\text{-}5)$$

$$L_r = 1.95r_{ts}\frac{E}{0.7F_y}\sqrt{\frac{Jc}{S_xh_o} + \sqrt{\left(\frac{Jc}{S_xh_o}\right)^2 + 6.76\left(\frac{0.7F_y}{E}\right)^2}} \qquad (5.8.3.1.3\text{-}6)$$

or conservatively, L_r can be taken as:

$$L_r = \pi r_{ts}\sqrt{\frac{E}{0.7F_y}} \qquad (5.8.3.1.3\text{-}7)$$

where:

C5.8.3.1.3

AISC (2011) does not provide formulas for lateral-torsional buckling resistance for HSS tubes. The reason is that for practical application in buildings, a deflection control will control before any LTB develops. However, for structural supports for signs, luminaires, and traffic signals, this may not be the case and the equations below may be used (see AISC (1993) Article F1.2a).

$$L_p = \frac{3750r_y}{F_yZ_x}\sqrt{JA} \qquad (C5.8.3.1.3\text{-}1)$$

$$L_r = \frac{57000r_y\sqrt{JA}}{F_yS_x} \qquad (C5.8.3.1.3\text{-}2)$$

$$r_{ts} = \sqrt{\frac{\sqrt{I_y C_w}}{S_x}} \quad (5.8.3.1.3\text{-}8)$$

$c = 1$ for doubly symmetric I-shapes

$$c = \frac{h_o}{2}\sqrt{\frac{I_y}{C_w}} \text{ for channels} \quad (5.8.3.1.3\text{-}9)$$

where:

L_b = length between points that are either braced against lateral displacement of compression flange or braced against twist of the cross section (in.),

L_p = the limiting laterally unbraced length for the limit state of yielding (in.),

L_r = the limiting laterally unbraced length for the limit state of inelastic LTB (in.),

h_o = the distance between flange centroids (in.),

C_w = warping constant (in.^{6}),

I_y = out-of-plane moment of inertia (in.^{4}),

S_x = elastic section modulus taken about the x axis (in.^{3}),

J = the torsional constant (in.^{4}),

F_y = yield stress (ksi),

Z_x = plastic section modulus taken about the x axis (in.^{3}), and

E = elastic modulus (ksi).

The moment gradient adjustment factor, C_b, is:

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} \quad (5.8.3.1.3\text{-}10)$$

where:

C_b can conservatively be taken as 1.0 and should be 1.0 for cantilever beams, and

M_{max} = absolute value of the maximum moment in the unbraced segment (kip-in.),

M_A = absolute value of the moment in the one-quarter point of the unbraced segment (kip-in.),

M_B = absolute value of the moment in the center of the unbraced segment (kip-in.),

M_C = absolute value of the moment in the three-quarter point of the unbraced segment (kip-in.).

5.8.3.2—Other I-Shaped Members with Compact or Noncompact Webs and Compact or Noncompact Flanges

The nominal bending strength shall be the lower value obtained according to the limit states of compression flange yielding, compression flange local buckling, tension flange yielding, and lateral-torsional buckling.

C5.8.3.2

Buckling limits ($\lambda > \lambda_r$) do not need to be considered because slender elements are not allowed.

5.8.3.2.1—Compression Flange Yielding

$$M_n = R_{pc} M_{yc} = R_{pc} F_y S_{xc} \qquad (5.8.3.2.1\text{-}1)$$

If $\frac{I_{yc}}{I_y} \leq 0.23$, use $R_{pc} = 1.0$

For compact web, $\lambda \leq \lambda_p$

$$R_{pc} = \frac{M_p}{M_{yc}} \qquad (5.8.3.2.1\text{-}2)$$

$$M_p = Z_x F_y \leq 1.65 S_{xc} F_y \qquad (5.8.3.2.1\text{-}3)$$

For noncompact web, $\lambda_p < \lambda \leq \lambda_r$

$$R_{pc} = \left[\frac{M_p}{M_{yc}} - \left(\frac{M_p}{M_{yc}} - 1\right)\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right)\right] \leq \frac{M_p}{M_{yc}} \qquad (5.8.3.2.1\text{-}4)$$

where:

S_{xc} = section modulus for the compression flange, (in[3]).

5.8.3.2.2—Compression Flange Local Buckling

If $\lambda \leq \lambda_p$ compression FLB buckling does not apply,

where $\lambda > \lambda_p$

$$M_n = \left[R_{pc} M_{yc} - \left(R_{pc} M_{yc} - F_L S_{xc}\right)\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right)\right] \qquad (5.8.3.2.2\text{-}1)$$

for:

$$\frac{S_{xt}}{S_{xc}} \geq 0.7, \quad F_L = 0.7F_y$$
$$\frac{S_{xt}}{S_{xc}} < 0.7, \quad F_L = \frac{S_{xt}}{S_{xc}} F_y \geq 0.5F_y \qquad (5.8.3.2.2\text{-}2)$$

5.8.3.2.3—Tension Flange Yielding

If $S_{xt} > S_{xc}$ tension flange yielding does not apply.

If $S_{xt} \leq S_{xc}$

$$M_n = R_{pt} M_{yt} = R_{pt} F_y S_{xt} \qquad (5.8.3.2.3\text{-}1)$$

For compact web, $\lambda \leq \lambda_p$

$$R_{pt} = \frac{M_p}{M_{yt}} \qquad (5.8.3.2.3\text{-}2)$$

For noncompact web, $\lambda > \lambda_p$

$$R_{pt} = \left[\frac{M_p}{M_{yt}} - \left(\frac{M_p}{M_{yt}} - 1\right)\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right)\right] \leq \frac{M_p}{M_{yt}} \qquad (5.8.3.2.3\text{-}3)$$

5.8.3.2.4—Lateral-Torsional Buckling

If $L_b \leq L_p$ LTB does not apply.

If $L_p \leq L_b \leq L_r$,

$$M_n = C_b\left[R_{pc} M_{yc} - \left(R_{pc} M_{yc} - F_L S_{xc}\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \leq R_{pc} M_{yc}$$

$$(5.8.3.2.4\text{-}1)$$

If $L_b > L_r$

$$M_n = F_{cr} S_{xc} \leq R_{pc} M_{yc} \qquad (5.8.3.2.4\text{-}2)$$

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_t}\right)^2} \sqrt{1 + 0.078 \frac{J}{S_{xc} h_o} \left(\frac{L_b}{r_t}\right)^2} \qquad (5.8.3.2.4\text{-}3)$$

$$L_p = 1.1 r_t \sqrt{\frac{E}{F_y}} \qquad (5.8.3.2.4\text{-}4)$$

$$J = 0 \text{ if } \frac{I_{yc}}{I_y} \leq 0.23 \qquad (5.8.3.2.4\text{-}5)$$

$$L_r = 1.95 r_t \frac{E}{F_y} \sqrt{\frac{J}{S_{xc} h_o} + \sqrt{\left(\frac{J}{S_{xc} h_o}\right)^2 + 6.76\left(\frac{F_y}{E}\right)^2}} \qquad (5.8.3.2.4\text{-}6)$$

The effective radius of gyration for LTB, r_t, can be approximated by the radius of gyration of the compression flange plus one-third of the compression portion of the web:

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1 + \frac{1}{6} a_w\right)}} \qquad (5.8.3.2.4\text{-}7)$$

where:

a_w = ratio of two times the web area in compression due to application of major axis bending moment alone to the area of the compression flange components.

5.8.4—Tees and Double Angles Loaded in Plane of Symmetry

The nominal bending strength shall be the lowest value obtained according to limit states of plastic moment, flange local buckling, stem local buckling, and lateral-torsional buckling.

5.8.4.1—Plastic Moment

$$M_n = M_p \qquad (5.8.4.1\text{-}1)$$

$M_p = Z_x F_y \leq 1.6 M_y$ for stems in tension

$M_n = M_y$ for stems in compression

5.8.4.2—Flange Local Buckling

If $\lambda \leq \lambda_p$ or flange in tension, then FLB does not apply.

If $\lambda > \lambda_p$ and flange in compression

$$M_n = M_p - \left(M_p - 0.7 F_y S_{xc}\right)\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right) \leq 1.6 M_y \qquad (5.8.4.2\text{-}1)$$

5.8.4.3—Stem Local Buckling

$$M_n = F_{cr} S_x \qquad (5.8.4.3\text{-}1)$$

For $\lambda < \lambda_p$ or stem in tension

$$F_{cr} = F_y \qquad (5.8.4.3\text{-}2)$$

For $\lambda \geq \lambda_p$ and stem in compression

$$F_{cr} = \left[2.55 - 1.84\frac{d}{t_w}\sqrt{\frac{F_y}{E}}\right]F_y \qquad (5.8.4.3\text{-}3)$$

5.8.4.4—Lateral-Torsional Buckling

$$M_n = M_{cr} = \frac{\pi\sqrt{EI_yGJ}}{L_b}\left[B + \sqrt{1+B^2}\right] \qquad (5.8.4.4\text{-}1)$$

where:

$B = 2.3\left(\frac{d}{L_b}\right)\sqrt{\frac{I_y}{J}}$ when stem in tension, and

$B = -2.3\left(\frac{d}{L_b}\right)\sqrt{\frac{I_y}{J}}$ when stem in compression and within the unbraced length.

5.8.5—I-Shaped, Channels, Tees, and Double Angles Bent about the Minor Axis

The nominal bending strength shall be the lower value obtained according to the limit states of plastic moment and flange local buckling.

5.8.5.1—Plastic Moment

$$M_n = M_p = Z_yF_y \leq 1.6S_yF_y \qquad (5.8.5.1\text{-}1)$$

5.8.5.2—Flange Local Buckling

If $\lambda \leq \lambda_p$ FLB does not apply.

If $\lambda > \lambda_p$

$$M_n = M_p - \left(M_p - 0.7F_yS_y\right)\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right) \qquad (5.8.5.2\text{-}1)$$

5.8.6—Single Angles

Single angles subject to flexure should be avoided unless loaded through the shear center. Single angles under flexure are subject to biaxial bending and torsion.

C5.8.6

If single angles are used in flexure, other resources such as AISC (2011) or LRFD Design should be consulted.

5.8.7—Round and Rectangular Bars

The nominal bending strength shall be the lower value obtained according to plastic moment and lateral-torsional buckling.

5.8.7.1—Plastic Moment

For round and rectangular bars bent about the minor axis, and rectangular bars with $\frac{L_b d}{t^2} \leq \frac{0.08E}{F_y}$ bent about the major axis:

$$M_n = M_p = F_y Z \leq 1.6 M_y \qquad (5.8.7.1\text{-}1)$$

5.8.7.2—Lateral-Torsional Buckling for Rectangular Bars Bent about the Major Axis

If $\frac{0.08E}{F_y} < \frac{L_b d}{t^2} \leq \frac{1.9E}{F_y}$

$$M_n = C_b \left[1.52 - 0.274 \left(\frac{L_b d}{t^2} \right) \frac{F_y}{\cancel{t}\ \underline{E}} \right] M_y \leq M_p \qquad (5.8.7.2\text{-}1)$$

If $\frac{L_b d}{t^2} > \frac{1.9E}{F_y}$

$$M_n = F_{cr} S_x \leq M_p \qquad (5.8.7.2\text{-}2)$$

$$F_{cr} = \frac{1.9 E C_b}{\frac{L_b d}{t^2}} \qquad (5.8.7.2\text{-}3)$$

where:

t = the section width (in.), and

d = the section depth (in.).

C5.8.7.2

For rectangular bars bent about the minor axis and for round bars, lateral-torsional buckling does not apply.

5.9—COMPONENTS IN TENSION

5.9.1—General

The provisions of this Article apply to tension of rolled open, tubular, and built-up plate sections.

The factored tensile resistance, P_r, shall be

$$P_r = \min\left[\phi_y P_{ny}, \phi_u P_{nu} \right] \qquad (5.9.1\text{-}1)$$

where:

ϕ_y and ϕ_u = resistance factor as specified in Article 5.5.3.2

C5.9.1

AISC (2011) design equations were incorporated for typical sign, luminaire, and signal supports. For members and limit states not addressed in these Specifications, other resources should be considered such as AISC (2011) and LRFD Design.

P_{nt} = minimum nominal tensile strength from the limit states of yield on the gross sectional area, and

P_{nu} = rupture strength on the effective net area.

5.9.2—Nominal Tensile Strength

The nominal tensile strength for yield on the gross section shall be:

$$P_{ny} = A_g F_y \qquad (5.9.2\text{-}1)$$

where:

A_g = the gross section area (in.^{2}), and

F_y = yield strength (ksi).

The tensile strength for the fracture on the effective net area shall be:

$$P_{nu} = A_e F_u \qquad (5.9.2\text{-}2)$$

where:

A_e = net effective area (in.^{2}), and

F_u = tensile strength (ksi).

C5.9.2

The definitions and limits on the effective net area are based on AISC (2011).

5.9.3—Effective Net Area

The effective net area, A_e, shall be taken equal to the net area, A_n, when the load is transmitted directly to each of the cross-sectional elements by bolts.

The net area, A_n, shall be calculated as the sum of the individual net areas along the potential critical section. When calculating, A_n, the width deducted for the bolt hole shall be taken as $^1/_{16}$ in. greater than the nominal dimension of the hole.

For a chain of holes across a part in any diagonal or zigzag line (staggered holes), the net width of the part shall be determined by deducting from the gross width the sum of the hole deductions for all the holes in the chain and adding, for each gauge space in the chain, the quantity $s^2/4g$.

For members without holes, the net area, A_n, is equal to the gross area, A_g.

When the load is transmitted through some but not all of the cross-sectional elements, shear lag shall be considered. The effective net area, A_e, shall be computed as:

$$A_e = UA_n \qquad (5.9.3\text{-}1)$$

where:

U = shear-lag reduction coefficient

C5.9.3

In lieu of the calculated value for U, the following values may be used for bolted connections:

U = 0.80 (single or double angles with four or more bolts per line in the direction of load)

For tension members, except plates and hollow structural shapes, connected by fasteners or longitudinal welds or with longitudinal welds in combination with transverse welds:

$$U = \left(1 - \frac{\bar{x}}{L}\right) \leq 0.9 \qquad (5.9.3\text{-}2)$$

where:

$\bar{x}$ = connection eccentricity, defined as the distance from the connection plane, or face of the member,

to the centroid of the section resisting the connection force, (in.)

L = length of connection in the direction of loading (in.)

U = 0.60 (single or double angles with three bolts per line in the direction of load)

U = 0.90 (with three or more bolts per line in the direction of load and $b_f \geq {}^{2}/_{3}d$)

U = 0.85 (flange connection I-shaped or tees with three or more bolts per line in the direction of load and $b_f < {}^{2}/_{3}d$)

Larger values of U are permitted to be used when justified by tests or other rational criteria.

For members connected by only transverse welds:

$$U = \frac{Area\ of\ Directly\ Connected\ Elements}{A_g}$$

For plate members connected by longitudinal welds along both edges:

for $L_w \geq 2w$, $U = 1.00$ (5.9.3-3)

for $2w > L_w \geq 1.5w$, then $U = 0.87$ (5.9.3-4)

for $1.5w > L_w \geq w$, then $U = 0.75$ (5.9.3-5)

where:

L_w = length of longitudinal weld (in.), and

w = plate width (distance between welds) (in.).

The effective net area, A_e, shall not be taken greater than 85 percent of the gross area, A_g, for the design of connecting elements such as splice plates, gusset plates, and connecting plates.

5.9.4—Slenderness Limit

For trusses, L/r shall not exceed 240 for members in tension.

5.10—COMPONENTS IN COMPRESSION

5.10.1—General

The provisions of this article apply to compression of rolled open, tubular, and built-up plate sections.

The factored compressive resistance, P_r, shall be:

C5.10.1

AISC (2011) design equations have been incorporated for typical sign, luminaire, and signal supports. For members and limit states not addressed in

$$P_r = \phi_c P_{nc} \quad (5.10.1\text{-}1)$$

where:

ϕ_c = resistance factor as specified in Article 5.5.3.2, and

P_{nc} = minimum nominal compressive strength defined in Article 5.10.2.

these Specifications, other resources should be considered such as AISC (2011) and LRFD Design.

5.10.2—Nominal Compressive Strength

5.10.2.1—Flexural Buckling

The nominal compressive strength shall be calculated as follows:

$$P_{nc} = A_g F_{cr} \quad (5.10.2.1\text{-}1)$$

when $\dfrac{KL}{r} \leq 4.71\sqrt{\dfrac{E}{QF_y}}$

$$F_{cr} = Q\left(0.658\right)^{\left(\frac{QF_y}{F_e}\right)} F_y \quad (5.10.2.1\text{-}2)$$

when $\dfrac{KL}{r} > 4.71\sqrt{\dfrac{E}{QF_y}}$

$$F_{cr} = 0.877 F_e \quad (5.10.2.1\text{-}3)$$

$$F_e = \frac{\pi^2 E}{\left(\dfrac{KL}{r}\right)^2} \quad (5.10.2.1\text{-}4)$$

If all section elements $\lambda \leq \lambda_r$

$$Q = 1.0 \quad (5.10.2.1\text{-}5)$$

C5.10.2.1

The radius of gyration, r, may be calculated at a distance of $0.50L$ for a tapered column. This value is conservative for all tapered light poles.

For a cantilever column, the effective length factor, K, may be taken as 2.1. Equations using Q are for members with slender elements (round and multi-sided tubular members). For other members, $Q = 1$ because slender elements are not allowed.

5.10.2.2—Round Tube with Slender Elements

If $\lambda > \lambda_r$

$$Q = 0.67 + \frac{0.038}{\left(\dfrac{D}{t}\right)}\left(\frac{E}{F_y}\right) \leq 1.0 \quad (5.10.2.2\text{-}1)$$

5.10.2.3—Multi-Sided and Square and Rectangular Tubes with Slender Elements

If $\lambda > \lambda_r$

$$Q = \frac{A_{EFF}}{A_g} \qquad (5.10.2.3\text{-}1)$$

A_{EFF} is calculated from the sum of parts using effective widths, b_e

$$b_e = 1.92t\sqrt{\frac{E}{f}}\left[1 - \frac{0.34}{\left(\frac{b}{t}\right)}\sqrt{\frac{E}{f}}\right] \leq b \qquad (5.10.2.3\text{-}2)$$

$f = F_{cr}$ using $Q = 1.0$

5.10.2.4—Single Angles

Single angles in trusses with the ratio of long-leg-width–short-leg-width less than 1.7 may be designed as axially loaded members with the following column slenderness ratios, KL/r.

For equal-leg angles or unequal-leg angles connected through the longer leg:

$$\frac{KL}{r} = 72 + 0.75\left(\frac{L}{r_x}\right)$$
$$\text{when } \frac{L}{r_x} \leq 80 \qquad (5.10.2.4\text{-}1)$$

$$\frac{KL}{r} = 32 + 1.25\left(\frac{L}{r_x}\right) \qquad (5.10.2.4\text{-}2)$$
$$\text{when } \frac{L}{r_x} > 80$$

For unequal-leg angles connected through the shorter leg, KL/r, from the applicable equation shall be increased by adding the amount:

$$4\left[\left(\frac{b_l}{b_s}\right)^2 - 1\right],$$

but KL/r shall not be taken less than $0.95KL/r_z$ where:

r_x = radius of gyration about the axis parallel to the connected leg (in.).

C5.10.2.4

If the longer-to-shorter-leg width ratio exceeds 1.7, the angle should be designed as a beam-column considering bending affects. The slenderness ratios indirectly account for bending in the angle due to eccentricity of the loading. The slenderness equations are from AISC (2011).

r_z = radius of gyration about the minor principle axis (in.),

b_l = longer leg width, (in.), and

b_s = shorter leg width, (in.)

5.10.2.5—Torsional Buckling

For non-doubly symmetric shapes and some doubly symmetric shapes, torsional and flexural-torsional buckling may need to be considered.

C5.10.2.5

Because torsional column buckling is not a common problem with sign and luminaire and signal supports members, strength equations are not included here. If torsional buckling is of concern, design equations of AISC (2011) should be applied.

5.10.3—Slenderness Limit

For trusses, KL/r shall not exceed 140 for members in compression.

5.11—COMPONENTS IN DIRECT SHEAR AND TORSION

5.11.1—General

The provisions of this article apply to direct shear and torsion of rolled open, tubular, and built-up plate sections.

The factored direct shear resistance, V_r, shall be:

$$V_r = \phi_v V_n \qquad (5.11.1\text{-}1)$$

And the factored torsional shear resistance, T_r, shall be:

$$T_r = \phi_t T_n \qquad (5.11.1\text{-}2)$$

where:

V_r = nominal direct shear capacity,

T_r = nominal torsion capacity, and

ϕ_v and ϕ_t = the resistance factor as specified in Article 5.5.3.2.

C5.11.1

Previous allowable stress design equations for typical sign, luminaire, and signal support structures were modified to limit state design equations for tubular members. AISC (2011) design equations have been incorporated for other typical structures. For structures not addressed in these Specifications, other resources should be considered such as AISC (2011) and LRFD Design.

Herein, direct shear is the shear force created by a change/gradient in bending moment.

5.11.2—Nominal Direct Shear Strength

The nominal direct shear strength due to shear shall be:

$$V_n = A_v F_{nv} \qquad (5.11.2\text{-}1)$$

where:

F_{nv} = nominal shear stress capacity (ksi), and

A_v = shear area (in.^{2}), as defined in Articles 5.11.2.1 and 5.11.2.2.

5.11.2.1—Nominal Shear Stress Capacity for Tubular Members

5.11.2.1.1—Round Tubular Members

The nominal shear stress capacity for round tubular shapes shall be the greater of:

$$F_{nv} = \frac{1.60E}{\sqrt{\frac{L_v}{D}}\left(\frac{D}{t}\right)^{5/4}} \qquad (5.11.2.1.1\text{-}1)$$

or

$$F_{nv} = \frac{0.78E}{\left(\frac{D}{t}\right)^{3/2}} \qquad (5.11.2.1.1\text{-}2)$$

but shall not exceed $0.6F_y$,

where:

L_v = distance from the maximum to zero shear force, and

$$A_v = \frac{A_g}{2} \qquad (5.11.2.1.1\text{-}3)$$

C5.11.2.1.1

AISC (2011) equations for nominal direct shear strength are incorporated for round tubes.

5.11.2.1.2—Multi-Sided Tubular Members

The nominal direct shear stress capacity for multi-sided ~~non-square and rectangular tubular shapes~~ tubes (not square or rectangular) shall be:

$$F_{nv} = 0.6F_y \qquad (5.11.2.1.2\text{-}1)$$

$$A_v = \frac{A_g}{2} \qquad (5.11.2.1.2\text{-}2)$$

C5.11.2.1.2

Previous editions of these specifications have shown that multi-sided tubes will not buckle under shear with width–thickness ratios limited to λ_{max}.

5.11.2.2—Nominal Direct Shear Strength for I-Shapes; Channels; Tees; and Square and Rectangular, and Double Angle Shapes

The nominal shear stress capacity shall be:

$$F_{nv} = 0.6F_y C_v \qquad (5.11.2.2\text{-}1)$$

If $\frac{h}{t} \le 1.10\sqrt{\frac{k_v E}{F_y}}$

$$C_v = 1.0 \qquad (5.11.2.2\text{-}2)$$

If $1.10\sqrt{\frac{k_v E}{F_y}} < \frac{b}{t} \leq 1.37\sqrt{\frac{k_v E}{F_y}}$

$$C_v = \frac{1.10\sqrt{\frac{k_v E}{F_y}}}{\frac{b}{t}} \qquad (5.11.2.2\text{-}3)$$

If $\frac{b}{t} > 1.37\sqrt{\frac{k_v E}{F_y}}$

$$C_v = \frac{1.51 k_v E}{\left(\frac{b}{t}\right)^2 F_y} \qquad (5.11.2.2\text{-}4)$$

For I-shapes, channels and tees:

$$A_v = A_w = dt_w \qquad (5.11.2.2\text{-}5)$$

$$k_v = 5 \qquad (5.11.2.2\text{-}6)$$

$$\frac{b}{t} = \frac{h}{t_w} \qquad (5.11.2.2\text{-}7)$$

For double angles:

$$A_v = 2bt \qquad (5.11.2.2\text{-}8)$$

$$k_v = 1.2 \qquad (5.11.2.2\text{-}9)$$

For square and rectangular shapes:

$$A_v = 2ht \qquad (5.11.2.2\text{-}10)$$

$$k_v = 5 \qquad (5.11.2.2\text{-}11)$$

$$\frac{b}{t} = \frac{h}{t} \qquad (5.11.2.2\text{-}12)$$

5.11.3—Nominal Torsion Strength

The nominal torsion strength due to torsion shall be:

$$T_n = C_t F_{nt} \qquad (5.11.3\text{-}1)$$

where:

T_n = nominal torsion strength, and

C_t = the torsional constant.

C5.11.3

Values for C_t are provided for different shapes in Appendix B.

5.11.3.1—Nominal Torsion Strength for Tubular Members

5.11.3.1.1—Round Tubular Members

The nominal torsion stress capacities for round tubular shapes shall be the greater of:

$$F_{nt} = \frac{1.23E}{\sqrt{\frac{L}{D}}\left(\frac{D}{t}\right)^{5/4}} \quad (5.11.3.1.1\text{-}1)$$

and

$$F_{nt} = \frac{0.6E}{\left(\frac{D}{t}\right)^{3/2}} \quad (5.11.3.1.1\text{-}2)$$

but shall not exceed $0.6F_y$.

5.11.3.1.2—Multi-Sided Tubular Members

The nominal torsion stress capacity for multi-sided ~~non-square and rectangular tubular shapes~~ tubes (not square or rectangular) shall be:

$$F_{nt} = 0.6F_y \quad (5.11.3.1.2\text{-}1)$$

C.5.11.3.1.2

Previous editions of these specifications have shown that multi-sided tubes will not buckle with width-to-thickness ratios limited to λ_{max}.

5.11.3.2—I-Shapes; Channels; Tees; and Square and Rectangular, and Angle Shapes

For torsion on open I-shape, channel, tee, and angle sections, AISC Design Guide 9 (1997) may be used to develop an appropriate nominal torsional capacity.

For square and rectangular shapes

$$F_{nt} = F_{nv} \quad (5.11.3.2\text{-}1)$$

5.12—COMBINED FORCES

5.12.1—Combined Force Interaction Requirements

Members subjected to combined bending, axial compression or tension, shear, and torsion shall be proportioned to meet the following:

$$\frac{P_u}{P_r} + \frac{BM_u}{M_r} + \left(\frac{V_u}{V_r} + \frac{T_u}{T_r}\right)^2 \le 1.0 \quad (5.12.1\text{-}1)$$

C5.12

AISC (2011) design equations were incorporated for typical sign, luminaire, and signal supports. For members and limit states not addressed in these specifications, other resources should be considered such as AISC (2011) and LRFD Design.

For structural supports for signs, luminaires, and traffic signals, direct shear is typically small and therefore only torsional effects are checked to determine which interaction equation to use.

If $\frac{T_u}{T_r} \leq 0.20$ torsional and shear effects can be ignored.

and when:

$$\frac{P_u}{P_r} \geq 0.20$$

$$\frac{P_u}{P_r} + \frac{8}{9}\frac{BM_u}{M_r} \leq 1.0 \quad (5.12.1\text{-}2)$$

when $\frac{P_u}{P_r} < 0.20$

$$\frac{P_u}{2P_r} + \frac{BM_u}{M_r} \leq 1.0 \quad (5.12.1\text{-}3)$$

For vertical luminaire supports, the term $\frac{P_u}{2P_r}$ may be approximated as 0.08.

For round and multi-sided tubular members,

$$M_u = \sqrt{M_{ux}^2 + M_{uy}^2} \quad (5.12.1\text{-}4)$$

and

$$V_u = \sqrt{V_{ux}^2 + V_{uy}^2} \quad (5.12.1\text{-}5)$$

For members with biaxial bending about geometric or principal axes, the term $\frac{BM_u}{M_r}$ may be expanded to:

$$B_x\frac{M_{ux}}{M_{rx}} + B_y\frac{M_{uy}}{M_{ry}} \quad (5.12.1\text{-}6)$$

$$\frac{V_u}{V_r} = \text{the greater of } \frac{V_{ux}}{V_{rx}} \text{ or } \frac{V_{uy}}{V_{ry}} \quad (5.12.1\text{-}7)$$

when member is in tension:

$$P_r = \phi_t P_{nt} \quad (5.12.1\text{-}8)$$

when member is in compression:

$$P_r = \phi_c P_{nc} \quad (5.12.1\text{-}9)$$

Moment Magnifier B:

For prismatic members:

For vertical luminaire supports, the term $\frac{P_u}{P_r}$ is relatively small where P_r is the buckling resistance of the entire pole subjected to the applied axial loads P_u. Poles are often tapered with multiple sections with different wall thicknesses; the axial loading typically consists of a concentrated luminaire load with a distributed dead load that is a function of the taper. Therefore, closed-form solutions are difficult. A rigorous numerical analysis to compute the buckling load may be employed and may be implemented in standard tools like a spreadsheet. The term $\frac{P_u}{P_r}$ is the inverse of the buckling load factor (include load and resistance factors).

If a finite element analysis (frame elements) is employed that considers only the second-order geometry sway effects (P-Δ), then the first term in Eq. 5.12.1-3 must be computed or approximated. If the finite element analysis considers both second-order sway effect and geometric axial "softening" effects of the stiffness matrix, then the $\frac{P_u}{P_r}$ term may be discarded as it is implicitly included in the analysis and will result in slightly higher bending moments. For simplicity, initial out-of-straightness, etc. does not need to be considered. Some commercial analysis programs properly consider both effects, others do not.

This includes square and rectangular tubes and other nontubular shapes.

Compression: $B = \dfrac{1}{1 - \dfrac{P_u}{P_e}}$ (5.12.1-10)

where:

$$P_e = \frac{\pi^2 E A_g}{\left(\dfrac{KL}{r}\right)^2} \quad (5.12.1\text{-}11)$$

Tension:

$$B = 1.0 \quad (5.12.1\text{-}12)$$

For non-prismatic members, Tension:

$$B = 1.0 \quad (5.12.1\text{-}13)$$

Compression: B shall be computed according to Section 4.

5.12.2—Bending of Square and Rectangular Tubes

Square and rectangular tubes shall meet the design requirements of Article 5.12.1 for bending about the geometric axes. In addition, this section applies to tubes bent about a skewed (diagonal) axis. The following interaction equation shall be satisfied:

$$\left(\frac{B_x M^*_{ux}}{M_{rx}}\right)^{\alpha} + \left(\frac{B_y M^*_{uy}}{M_{ry}}\right)^{\alpha} \leq 1 \quad (5.12.2\text{-}1)$$

where for tubes with $\lambda \leq \lambda_p$

$\alpha = 1.60, M_{rx} = \phi_f M_{px}, M_{ry} = \phi_f M_{py}$

and, for tubes with $\lambda_r < \lambda \leq \lambda_{max}$

$\alpha = 1.00, M_{rx} = \phi_f M_{nx}$ and $M_{ry} = \phi_f M_{ny}$

M^*_{ux}, M^*_{uy} = factored moments from skewed diagonal loading.

C5.12.2

NCHRP Report 494, *Supports for Highway Signs, Luminaries, and Traffic Signals* (Fouad et al., 2003) compared theoretical diagonal bending to experimental tests. The interaction increase in nominal strength is justified for tubes bent about the diagonal for sections with limited width-thickness ratios. Although the diagonal strength properties are significantly less than the primary axis properties, tests show additional strength compared with current strength predictions. For compact sections, the reserve strength is 33 percent higher for bending about a diagonal axis ($Z_x/S_x = 1.5$) than about the principal axes ($Z_x/S_x = 1.13$), where Z_x and S_x are the plastic and elastic section moduli, respectively.

5.13—CABLES AND CONNECTIONS

The provisions of this Article apply to cables and their connections.

The factored tensile resistance, R_{rt}, shall be

$$R_{rt} = \phi_{rt} R_{nr} \quad (5.13\text{-}1)$$

where:

C5.13

Typically manufacturers' data may be used for the resistances.

ϕ_n is the resistance factor as specified in Article 5.5.3.2.

For horizontal supports (wire and connections) of span-wire pole structures, the resistance of the cable or connection is the nominal breaking strength of the cable or connection.

5.14—WELDED CONNECTIONS

Welding design and fabrication shall be in accordance with the latest edition of the *AWS Structural Welding Code—Steel* (2010) and *AWS Structural Welding Code—Reinforcing Steel* (2011).

Fatigue considerations are provided in Section 11.

C5.14

Hybrid laser arc welding (HLAW) is categorized in AWS D1.1 as "Other Welding Processes." Process variables are to be agreed upon by the Fabricator and Owner. Fabrication guidance is provided in Division II.

5.15—BOLTED CONNECTIONS

Design of bolted connections shall be in accordance with the current LRFD Design.

Fatigue considerations are provided in Section 11.

5.16—ANCHOR BOLT CONNECTIONS

This Article provides the minimum requirements for design of steel anchor bolts used to transmit loads from attachments into concrete supports or foundations by means of tension, bearing, and shear. A minimum of eight anchor bolts shall be used to connect high-mast lighting towers.

C5.16

Figure C5.16-1 shows a typical steel-to-concrete double-nut connection. Figure C5.16-2 shows a typical single-nut connection. Installation considerations are provided in Division III.

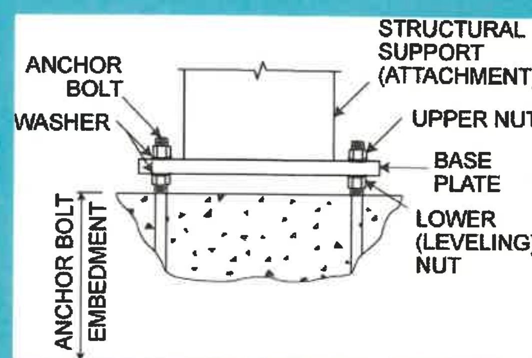


Figure C5.16-1—Typical Double-Nut Connection

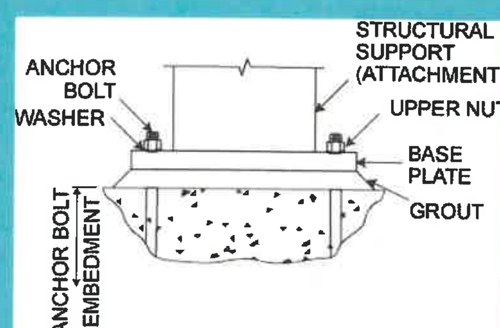


Figure C5.16-2. Typical Single-Nut Connection

5.16.1—Anchor Bolt Types

Cast-in-place anchor bolts shall be used in new construction.

The following requirements shall apply:

- Anchor bolts may be headed through the use of a preformed bolt head or by other means, such as a nut, flat washer, or plate;
- Hooked anchor bolts with a yield strength not exceeding 55 ksi may be used: and
- Deformed reinforcing bars may be used as anchor bolts.

C5.16.1

The ring-shaped base plate of a high-level (pole-type) luminaire support has low bending stiffness. The number of anchor bolts and the geometry of the base plate determine the stiffness of the base plate. Research, both fatigue tests and analytical studies, indicates that using less than 12 bolts can result in a reduction in fatigue performance in some connections. The fatigue strength of the butt-welded connection detail with an external collar reinforcement shown in Detail 4.8 of Table 11-9.3.1-1 is less sensitive to the number of anchor bolts and as few as 8 bolts can be used with this detail. However, due to the field problems in properly tightening the anchor bolts, the use of 12 bolts is recommended to provide adequate anchorage stiffness when fatigue is controlling the design of the luminaire support connection.

Research (Jirsa et al., 1984) has shown that headed cast-in-place anchor bolts perform significantly better than hooked anchor bolts, regarding possible pull-out prior to development of full tensile strength. Caution should be exercised when using deformed reinforcing bars as anchor bolts, because no fatigue test data are available on threaded reinforcing bar. The ductility of deformed reinforcing bars, as measured by elongation, can be significantly less than most other anchor bolts.

Anchor bolts with hooks make it impossible to perform a proper ultrasound inspection.

5.16.2—Anchor Bolt Materials

Anchor bolt material not otherwise specified shall conform to the requirements of ASTM F1554.

For hooked smooth bars, the yield strength shall not exceed 55 ksi.

Reinforcing bar material used for anchor bolts shall conform to ASTM A615 or ASTM A706. The yield strength shall not exceed 80 ksi.

Typical anchor bolt design material properties are provided in Table 5.16.2-1.

C5.16.2

Steel with yield strengths greater than 120 ksi have been found to be susceptible to stress corrosion in most anchorage environments [ACI 349–90 (1995)]. Galvanized steel with tensile strengths greater than 160 ksi are more susceptible to hydrogen embrittlement.

Threaded reinforcing bars (ASTM A706) may be used for anchor bolts. Reinforcing bars conforming to ASTM A615 have been used in the past. However, because of possible low toughness, ASTM A615 reinforcing bars should not be used for nonredundant, fatigue susceptible support structures such as cantilevers and high-mast luminaries. Anchor bolts conforming to ASTM F1554 usually have satisfactory fracture toughness. Charpy V-notch impact testing is not required for anchor bolt material.

Table 5.16.2-1—Typical Anchor Bolt Material

Material Specification	Yield Strength, ksi	Minimum Tensile Strength, ksi
ASTM F1554 Bolts	36	58
ASTM F1554 Bolts	55	75
ASTM F1554 Bolts	105	125
ASTM A706 Bars	60	80

Note: ASTM A615 bars are not recommended for anchor bolts when subject to fatigue.

5.16.3—Design Basis

The anchor bolts and their anchorage shall be designed to transmit loads from the attachment into the concrete support or foundation by means of tension, bearing, and shear, or any combination thereof.

The design of the anchor bolt and its anchorage shall ensure transfer of load from anchor to concrete. The anchorage system shall be proportioned such that the load in the steel portion of the anchorage will reach its minimum tensile strength prior to failure of the concrete.

C5.16.3

AISC (2006), *Design Guide 1: Base Plate and Anchor Rod Design* may be used. Concrete anchorages may be designed using ACI 318-11, Appendix D. Resistance factors shall be as specified in ACI 318-11. Loads shall be determined from this Specification.

A ductile connection to concrete fails by yielding of the steel anchor. A nonductile failure will occur by a brittle fracture of the concrete in tension or by the anchor slipping in the concrete without the steel yielding. All anchor bolts should be designed for a ductile steel failure prior to any sudden loss of capacity of the anchorages resulting from a brittle failure of the concrete.

5.16.3.1—Double-Nut Connections

For double-nut connections, when clearance between the bottom of the leveling nuts and the top of the concrete foundation is less than or equal to one bolt diameter, bending stresses in the anchor bolts may be ignored.

C5.16.3.1

The AISC (2006) *Design Guide 1: Base Plate and Anchor Rod Design* does not explicitly address the exclusion of anchor bolt bending for situations with minimal gap between the lower leveling nut and the concrete for double-nut connections. The previous editions of the *AASHTO LTS Design Specifications* included the design clarification that the bending stresses shall be considered when the gap between the leveling nut and the concrete is greater than one bolt diameter. By including the bending stress in the calculations, structures with a high torsion load, such as a cantilever mast arm or sign structure, will result in a significant increase in anchorage capacity required. Decades of in-field applications designed without considering bending stress have not resulted in documented issues relating to the anchor bolts. Based on past experience, the clarification to exclude bending stresses with minimal gap between the lower leveling nut and the concrete, for double-nut connections, has been included. Recent research by Cook et al. provides input on the impact of bending stresses on anchor bolts for the Designer's further review.

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SECTION 10:

SERVICEABILITY REQUIREMENTS

10.1—SCOPE

This Section provides serviceability requirements for support structures.

10.2—DEFINITIONS

Camber—The condition of the horizontal support being arched.

Quadri-Chord Truss—A horizontal member composed of four longitudinal chords connected by bracing.

Rake—To slant or incline from the vertical.

Tri-Chord Truss—A horizontal member composed of three longitudinal chords connected by bracing.

10.3—NOTATION

E = modulus of elasticity (ksi) (C10.5)

H = height of vertical support (in.) (C10.5)

I = moment of inertia of vertical support (in.^{4}) (C10.5)

L = distance between supports for an overhead bridge structure; distance from vertical support to free end for horizontal cantilevered support (in.) (10.4.1) (C10.4.1) (10.4.3.1) (10.5) (C10.5)

M = moment caused by dead loads applied to the vertical support at the connection of the horizontal support (lb-in.) (C10.5)

r = radius of gyration (in.) (10.4.3.1)

u = prefabricated camber (slope) in the horizontal cantilevered arm (in./in.) (C10.5)

δ_{DL} = deflection at free end of horizontal support under dead load (in.) (C10.5)

δ_P = deflection at tip of vertical support under dead load from horizontal cantilevered support (in.) (C10.5)

δ_{PDL} = deflection at free end of horizontal support caused by slope at the tip of the vertical support (in.) (C10.5)

δ_{TOTAL} = total dead load deflection at free end of horizontal support (in.) (C10.5)

θ = rotation at the top of the pole (radians) (C10.5)

10.4—DEFLECTION

Highway support structures of all materials should be designed to have adequate structural stiffness that will result in acceptable serviceability performance. Deflections for specific structure types shall be limited as provided in Articles 10.4.1 and 10.4.2. Permanent camber for specific structure types shall be provided per Article 10.5.

C10.4

The deflection limits that are set by these Specifications are to serve two purposes. The first purpose is to provide an aesthetically pleasing structure under dead load conditions. The second purpose is to provide adequate structural stiffness that will result in acceptable performance under applied loads.

10.4.1—Overhead Bridge Supports for Signs and Traffic Signals

For overhead bridge monotube and truss structures supporting signs and traffic signals, the maximum vertical deflection of the horizontal support resulting from Service I load combination shall be limited to $L/150$, where L is the span length.

C10.4.1

Research was sponsored by the Arizona Department of Transportation (Ehsani et al., 1984; Martin et al., 1985) to determine an appropriate deflection limitation for steel monotube bridge support structures. This research included field tests and analytical studies using computer modeling.

monotube bridge structures. The 1989 Interim Specifications were revised to limit deflection to the span divided by 150 for dead and ice load applications based on this research.

A later study (Lundgren, 1989) indicated that because the deflection criterion was an aesthetic limitation, it could be increased to the span divided by 100; however, no additional work has been found to justify changing the deflection limit to a more liberal value. Although this study considered only steel members, the deflection limit has been generalized for other materials because aesthetics was the governing consideration.

Other types of overhead bridge sign supports (i.e., two-chord, tri-chord, and quadri-chord trusses) generally have higher stiffness than the monotube type. A dead load deflection limit of the span divided by 150 (i.e., $L/150$) may be adopted as a conservative limit for those types of overhead bridge sign and traffic signal support structures made with two-chord, tri-chord, or quadri-chord trusses.

10.4.2—Cantilevered Supports for Signs, Luminaires, and Traffic Signals

10.4.2.1—Vertical Supports

The horizontal deflection limits for vertical supports, such as street lighting poles, traffic signal structures, and sign structures, shall be as follows:

- Under Service I II load combination, the deflection at the top of vertical supports with transverse load applications shall be limited to 2.5 percent of the structure height; and
- Under Service I II load combination, the slope at the top of vertical supports with moment load applications shall be limited to 0.35 in./ft.

For luminaire support structures under Service I load combination (i.e., dead load and wind), deflection shall be limited to 10 percent of the structure height.

Deflections shall be computed by usual methods or equations for elastic deflections. For prestressed concrete members, the effects of cracking and reinforcement on member stiffness shall be considered.

C10.4.2.1

The dead load deflection and slope limitations were developed based on aesthetic considerations. The 2.5 percent deflection limit was developed for transverse load applications, such as strain pole applications, where a dead load caused by span-wire tension could cause unsightly deflection. The horizontal linear displacement at the top of the structure is measured in relation to a tangent to the centerline at the structure's base. The slope limitation of 0.35 in./ft, which is equivalent to an angular rotation of 1 degree-40 minutes, was initially developed for street lighting poles with a single mast arm, where the mast arm applied a concentrated dead load moment that could also cause unsightly deflections. It is measured by the angular rotation of the centerline at the top of the structure in relation to the centerline at its base. The concentrated moment loads result from the effect of eccentric loads of single or unbalanced multiple horizontally mounted arm members and their appurtenances.

The 10 percent deflection limitation for the Service I load combination constitutes a safeguard against the design of highly flexible structures. It is intended mainly for high-level lighting poles. The deflections are calculated with the unit load factors defined in Article 3.4, and second-order effects are normally considered in the analysis.

10.4.2.2—Horizontal Supports

Adequate stiffness shall be provided for the horizontal supports of cantilevered sign and traffic signal structures that will result in acceptable serviceability performance.

Galloping and truck gust-induced vibration deflections of cantilevered single-arm sign supports and traffic signal

C10.4.2.2

No dead load deflection limit is prescribed for horizontal supports of cantilevered sign and traffic signal structures. Stiffness requirements are determined by the Designer. Structures are typically raked or the horizontal supports are cambered such that the dead load deflection at the end of the arm is above a horizontal reference. Camber requirements for

Appendix B:

Design Aids

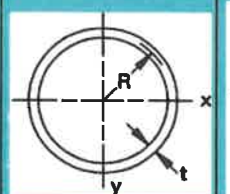
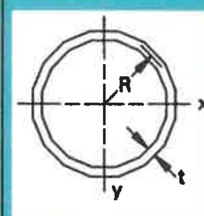
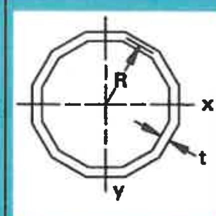
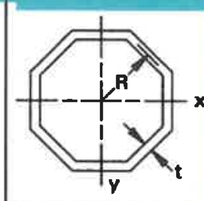
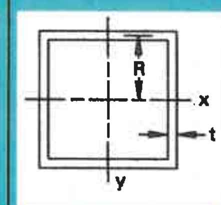
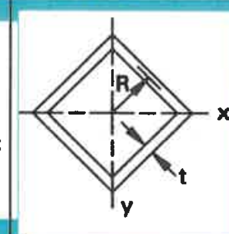
B.1—NOTATION

A = area (B.2) (B.3) (in.^{2})
C = cross-sectional constant (B.2) (B.4)
C_t = torsional constant (B.3) (in.^{3})
d_a = diameter at free end (B.5) (in.)
d_b = diameter at fixed end (B.5) (in.)
E = modulus of elasticity (B.4) (B.5) (ksi)
f_b = bending stress (ksi) (B.3)
f_{vb} = shear stress due to transverse loads (B.3) (ksi)
f_{vt} = shear stress due to torsion (B.3) (ksi)
f_x = stress due to bending about the x-axis (B.3) (ksi)
f_y = stress due to bending about the y-axis (B.3) (ksi)
I = moment of inertia (in.^{4}) (B.2)
I_a = moment of inertia of cross-section at free end of beam (B.5) (in.^{4})
K_p = shape factor (B.2)
k_t = stress concentration factor (B.3)
L = length of the beam (B.4) (B.5) (in.)
M = applied moment (B.4) (B.5) (kip-in.)
M_z = torsional moment (B.3) (kip-in.)
n' = ratio of the inside-corner radius to wall thickness (B.3)
P = horizontal load applied at free end of beam (B.5) (kip)
R = radius measured to the mid-thickness of the wall (B.2) (B.3) (in.)
R_A = radius measured to mid-thickness of wall at free end (B.4) (in.)
R_B = radius measured to mid-thickness of wall at fixed end (B.4) (in.)
S = section modulus (B.2) (B.3) (in.^{3})
t = wall thickness (B.2) (B.3) (B.4) (in.)
V_s = applied shear (B.3) (kip)
W = load (B.4) (kip)
w = load per unit length (B.4) (B.5) (kip/in.)
θ_{max} = maximum slope at free end of beam (rad) (B.4) (B.5) (radians)
Z = plastic section modulus (B.2) (in.^{3})
y_{max} = maximum horizontal deflection at free end of beam (B.4) (B.5) (in.)

B.2—SECTIONAL PROPERTIES FOR TUBULAR SHAPES

Table B.2-1 provides approximate equations to compute sectional properties of tubular shapes.

Table B.2-1—Estimated Sectional Properties for Common Tubular Shapes

Property	Round Tube	Hexdecagonal Tube	Dodecagonal Tube	Octagonal Tube	Square Tube	Square Tube (Axis on Diagonal)
Moment of inertia, I	$3.14R^3t$	$3.22R^3t$	$3.29R^3t$	$3.50R^3t$	$5.33R^3t$	$5.33R^3t$
Section modulus, S	$3.14R^2t$	$3.22R^2t$	$3.29R^2t$	$3.50R^2t$	$5.33R^2t$	$3.77R^2t$
Area, A	$6.28Rt$	$6.37Rt$	$6.43Rt$	$6.63Rt$	$8.00Rt$	$8.00Rt$
Shape factor, $K_p = Z/S$	1.27	1.27	1.26	1.24	1.12	$\cancel{1.5}$
Radius of gyration, $\cancel{R}/r$	$0.707R$	$0.711R$	$0.715R$	$0.727R$	$0.816R$	$0.816R$
Cross-sectional constant, C	3.14	3.22	3.29	3.50	5.33	$\cancel{5.33}$
Pictorial representation						

Notations:

C = cross-sectional constant used in Table B.4-1

R = radius measured to the mid-thickness of the wall

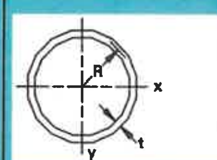
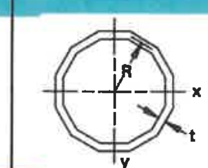
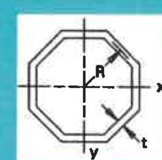
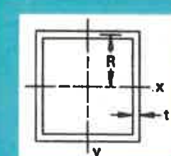
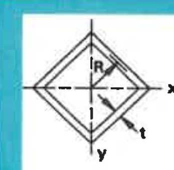
t = wall thickness

Z = plastic section modulus

B.3—STRESSES FOR TUBULAR SECTIONS

Table B.3-1 provides equations to calculate stresses for round and multisided tubular shapes. If more than one equation is provided, then the largest equation result should be used. For round tubes in biaxial bending, the maximum bending stress occurs at the resulting moment. For multisided tubular shapes in biaxial bending, the maximum bending stress occurs at one of the corners. The maximum transverse shear stress for tubular shape occurs in the tube walls that are intersected by the neutral axis, which is normal to the resultantant shear force.

Table B.3-1—Formulas for Maximum Stresses in Common Tubular Shapes

Stress	Round Tube	Hexdecagonal Tube	Dodecagonal Tube	Octagonal Tube	Square Tube	Square Tube (Axis on Diagonal)
Maximum bending stress, f_b	$\sqrt{f_x^2 + f_y^2}$	$0.199 f_x + f_y$ or $0.567 f_x + 0.848 f_y$ or $0.848 f_x + 0.567 f_y$ or $f_x + 0.199 f_y$	$0.732(f_x + f_y)$ or $f_x + 0.268 f_y$ or $0.268 f_x + f_y$	$f_x + 0.414 f_y$ or $0.414 f_x + f_y$	$f_x + f_y$	f_x or f_y
Maximum shear stress due to transverse loads, f_{vb}	$\frac{2.0V_x}{A}$	$\frac{2.02V_x}{A}$	$\frac{2.025V_x}{A}$	$\frac{2.05V_x}{A}$	$\frac{2.25V_x}{A}$	$\frac{2.12V_x}{A}$
Maximum shear stress due to torsion, f_{vt}	$\frac{M_z}{6.28R^2t}$	$\frac{M_z k_t}{6.37R^2t}$	$\frac{M_z k_t}{6.43R^2t}$	$\frac{M_z k_t}{6.63R^2t}$	$\cancel{\frac{M_z k_t}{8.00R^2}s}$ $\frac{M_z k_t}{8.00R^2t}$	$\frac{M_z k_t}{8.00R^2t}$
Torsional constant for stress Computations, e.g., C_t in Article 5.11.3.	$6.28R^2t$	$\frac{6.37R^2t}{k_t}$	$\frac{6.43R^2t}{k_t}$	$\frac{6.63R^2t}{k_t}$	$\cancel{\frac{8.00R^2t}{k_t s}}$ $\frac{8.00R^2t}{k_t}$	$\cancel{\frac{8.00R^2t}{k_t s}}$ $\frac{8.00R^2t}{k_t}$
Values of k_t, stress concentration factor	See Figure B.3-1					
Pictorial representation						

Notation:

A = area
f_{vb} = shear stress due to transverse loads
f_{vt} = shear stress due to torsion
f_x = stress due to bending about the x-axis
f_y = stress due to bending about the y-axis
k_t = stress concentration factor for multisided shapes
M_z = torsional moment
R = radius measured to the mid-thickness of the wall
t = wall thickness
V_x = applied shear

Torsional shear stresses are present on a transverse crosssection due to torsional moments, M_z, and are uniform around the periphery of thin-walled tubular sections except for stress concentrations at the corners of polygonal tubes. The stress concentration factor for a polygonal tube is obtained from Timoshenko's *Strength of Materials* (1957). The stress concentration factor may be found by using Figure B.3-1 or by using the following equation:

$$k_t = \frac{t}{R}\left[\frac{\dfrac{R}{n't} - \dfrac{1}{2}\left(1 + \dfrac{n'+1}{n'}\right)}{\ln\left(\dfrac{n'+1}{n'}\right)}\right] + \frac{n't}{R} \geq 1.0 \qquad \text{(B.3-1)}$$

where (consistent units):

k_t = stress concentration factor

t = wall thickness

R = radius measured to the mid-thickness of the wall

n' = ratio of the inside-corner radius to wall thickness

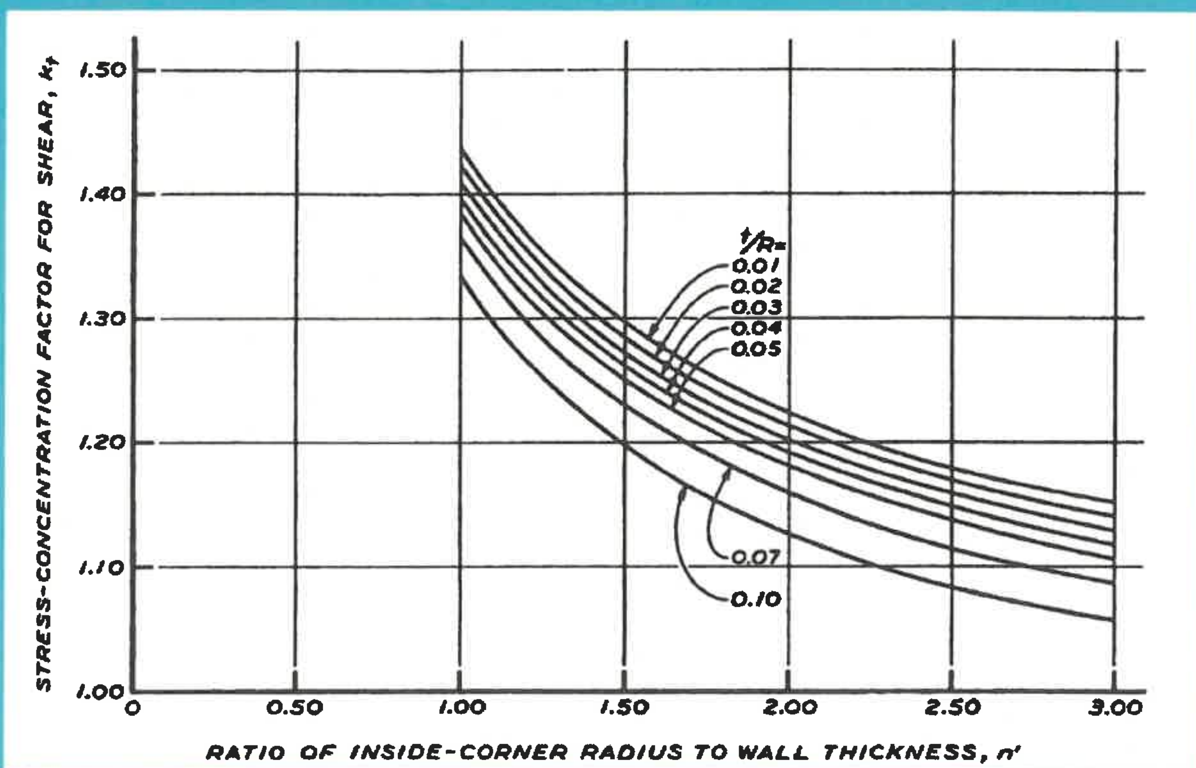


Figure B.3-1—Stress-Concentration Factors for Polygonal Tubes in Torsion

Source: Timoshenko, 1957.