2. Slender Stiffened Elements, Q_a

The reduction factor, Q_a , for slender *stiffened elements* is defined as follows:

$$Q_a = \frac{A_e}{A_g} \quad \text{(E7-16)}$$

where

A_g = gross cross-sectional area of member, in.^{2} (mm^2)

A_e = summation of the effective areas of the cross section based on the reduced *effective width*, b_e, in.^{2} (mm^2)

The reduced effective width, b_e, is determined as follows:

(a) For uniformly compressed slender elements, with $\frac{b}{t} \geq 1.49\sqrt{\frac{E}{f}}$, except flanges of square and rectangular sections of uniform thickness:

$$b_e = 1.92t\sqrt{\frac{E}{f}}\left[1 - \frac{0.34}{(b/t)}\sqrt{\frac{E}{f}}\right] \leq b \quad \text{(E7-17)}$$

where

f is taken as F_{cr} with F_{cr} calculated based on $Q = 1.0$

(b) For flanges of square and rectangular *slender-element sections* of uniform thickness with $\frac{b}{t} \geq 1.40\sqrt{\frac{E}{f}}$:

$$b_e = 1.92t\sqrt{\frac{E}{f}}\left[1 - \frac{0.38}{(b/t)}\sqrt{\frac{E}{f}}\right] \leq b \quad \text{(E7-18)}$$

where

$f = P_n/A_e$

User Note: In lieu of calculating $f = P_n/A_e$, which requires iteration, f may be taken equal to F_y. This will result in a slightly conservative estimate of *column available strength*.

(c) For axially loaded circular sections:

When $0.11\frac{E}{F_y} < \frac{D}{t} < 0.45\frac{E}{F_y}$

$$Q = Q_a = \frac{0.038E}{F_y(D/t)} + \frac{2}{3} \quad \text{(E7-19)}$$

where

D = outside diameter of round *HSS*, in. (mm)

t = thickness of wall, in. (mm)

CHAPTER F

DESIGN OF MEMBERS FOR FLEXURE

This chapter applies to members subject to simple bending about one principal axis. For simple bending, the member is loaded in a plane parallel to a principal axis that passes through the shear center or is restrained against twisting at *load* points and supports.

The chapter is organized as follows:

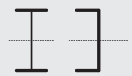
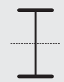
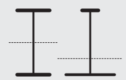
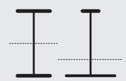
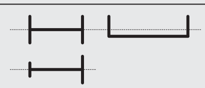
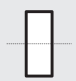
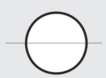
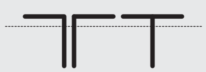
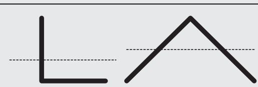
F1. General Provisions
F2. Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis
F3. Doubly Symmetric I-Shaped Members with Compact Webs and Noncompact or Slender Flanges Bent About Their Major Axis
F4. Other I-Shaped Members With Compact or Noncompact Webs Bent About Their Major Axis
F5. Doubly Symmetric and Singly Symmetric I-Shaped Members With Slender Webs Bent About Their Major Axis
F6. I-Shaped Members and Channels Bent About Their Minor Axis
F7. Square and Rectangular HSS and Box-Shaped Members
F8. Round HSS
F9. Tees and Double Angles Loaded in the Plane of Symmetry
F10. Single Angles
F11. Rectangular Bars and Rounds
F12. Unsymmetrical Shapes
F13. Proportions of Beams and Girders

User Note: For cases not included in this chapter the following sections apply:

- Chapter G — Design provisions for shear
- H1–H3 — Members subject to biaxial flexure or to combined flexure and axial force
- H3 — Members subject to flexure and torsion
- Appendix 3 — Members subject to *fatigue*

For guidance in determining the appropriate sections of this chapter to apply, Table User Note F1.1 may be used.

TABLE USER NOTE F1.1
Selection Table for the Application of Chapter F Sections

Section in Chapter F	Cross Section	Flange Slenderness	Web Slenderness	Limit States
F2		C	C	Y, LTB
F3		NC, S	C	LTB, FLB
F4		C, NC, S	C, NC	Y, LTB, FLB, TFY
F5		C, NC, S	S	Y, LTB, FLB, TFY
F6		C, NC, S	N/A	Y, FLB
F7		C, NC, S	C, NC	Y, FLB, WLB
F8		N/A	N/A	Y, LB
F9		C, NC, S	N/A	Y, LTB, FLB
F10		N/A	N/A	Y, LTB, LLB
F11		N/A	N/A	Y, LTB
F12	Unsymmetrical shapes, other than single angles	N/A	N/A	All limit states

Y = yielding, LTB = lateral-torsional buckling, FLB = flange local buckling, WLB = web local buckling, TFY = tension flange yielding, LLB = leg local buckling, LB = local buckling, C = compact, NC = noncompact, S = slender

F1. GENERAL PROVISIONS

The *design flexural strength*, $\phi_b M_n$, and the *allowable flexural strength*, M_n/Ω_b, shall be determined as follows:

(1) For all provisions in this chapter

$$\phi_b = 0.90 \text{ (LRFD)} \qquad \Omega_b = 1.67 \text{ (ASD)}$$

and the *nominal flexural strength*, M_n, shall be determined according to Sections F2 through F13.

(2) The provisions in this chapter are based on the assumption that points of support for *beams* and girders are restrained against rotation about their longitudinal axis.

(3) For singly symmetric members in *single curvature* and all doubly symmetric members:

C_b, the *lateral-torsional buckling* modification factor for nonuniform moment diagrams when both ends of the segment are braced is determined as follows:

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} \qquad \text{(F1-1)}$$

where

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

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

M_B = absolute value of moment at centerline of the unbraced segment, kip-in. (N-mm)

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

For cantilevers or overhangs where the free end is unbraced, $C_b = 1.0$.

User Note: For doubly symmetric members with no transverse loading between brace points, Equation F1-1 reduces to 1.0 for the case of equal end moments of opposite sign (uniform moment), 2.27 for the case of equal end moments of the same sign (*reverse curvature* bending), and to 1.67 when one end moment equals zero. For singly symmetric members, a more detailed analysis for C_b is presented in the Commentary.

(4) In singly symmetric members subject to reverse curvature bending, the *lateral-torsional buckling strength* shall be checked for both flanges. The available flexural strength shall be greater than or equal to the maximum required moment causing compression within the flange under consideration.

F2. DOUBLY SYMMETRIC COMPACT I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MAJOR AXIS

This section applies to doubly symmetric I-shaped members and channels bent about their major axis, having compact webs and compact flanges as defined in Section B4.1 for flexure.

User Note: All current ASTM A6 W, S, M, C and MC shapes except W21×48, W14×99, W14×90, W12×65, W10×12, W8×31, W8×10, W6×15, W6×9, W6×8.5 and M4×6 have compact flanges for F_y = 50 ksi (345 MPa); all current ASTM A6 W, S, M, HP, C and MC shapes have compact webs at $F_y \le 65$ ksi (450 MPa).

The *nominal flexural strength*, M_n, shall be the lower value obtained according to the *limit states* of *yielding (plastic moment)* and *lateral-torsional buckling*.

1. Yielding

$$M_n = M_p = F_y Z_x \quad \text{(F2-1)}$$

where

F_y = *specified minimum yield stress* of the type of steel being used, ksi (MPa)
Z_x = plastic section modulus about the *x*-axis, in.^{3} (mm^3)

2. Lateral-Torsional Buckling

(a) When $L_b \le L_p$, the *limit state* of *lateral-torsional buckling* does not apply.
(b) When $L_p < L_b \le L_r$

$$M_n = C_b\left[M_p - \left(M_p - 0.7F_yS_x\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p \quad \text{(F2-2)}$$

(c) When $L_b > L_r$

$$M_n = F_{cr}S_x \le M_p \quad \text{(F2-3)}$$

where

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

$$F_{cr} = \frac{C_b\pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2}\sqrt{1 + 0.078\frac{Jc}{S_xh_o}\left(\frac{L_b}{r_{ts}}\right)^2} \quad \text{(F2-4)}$$

and where

E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)
J = torsional constant, in.^{4} (mm^4)
S_x = elastic section modulus taken about the *x*-axis, in.^{3} (mm^3)
h_o = distance between the flange centroids, in. (mm)

User Note: The square root term in Equation F2-4 may be conservatively taken equal to 1.0.

User Note: Equations F2-3 and F2-4 provide identical solutions to the following expression for lateral-torsional buckling of doubly symmetric sections that has been presented in past editions of the AISC LRFD Specification:

$$M_{cr} = C_b\frac{\pi}{L_b}\sqrt{EI_yGJ + \left(\frac{\pi E}{L_b}\right)^2 I_yC_w}$$

The advantage of Equations F2-3 and F2-4 is that the form is very similar to the expression for lateral-torsional buckling of singly symmetric sections given in Equations F4-4 and F4-5.

The limiting lengths L_p and L_r are determined as follows:

$$L_p = 1.76r_y\sqrt{\frac{E}{F_y}} \quad \text{(F2-5)}$$

$$L_r = 1.95r_{ts}\frac{E}{0.7F_y}\sqrt{\frac{Jc}{S_xh_o} + \sqrt{\left(\frac{Jc}{S_xh_o}\right)^2 + 6.76\left(\frac{0.7F_y}{E}\right)^2}} \quad \text{(F2-6)}$$

where

$$r_{ts}^2 = \frac{\sqrt{I_yC_w}}{S_x} \quad \text{(F2-7)}$$

and the coefficient c is determined as follows:

(a) For doubly symmetric I-shapes: $c = 1$ (F2-8a)

(b) For channels: $c = \frac{h_o}{2}\sqrt{\frac{I_y}{C_w}}$ (F2-8b)

User Note: For doubly symmetric I-shapes with rectangular flanges, $C_w = \frac{I_yh_o^2}{4}$ and thus Equation F2-7 becomes

$$r_{ts}^2 = \frac{I_yh_o}{2S_x}$$

r_{ts} may be approximated accurately and conservatively as the radius of gyration of the compression flange plus one-sixth of the web:

$$r_{ts} = \frac{b_f}{\sqrt{12\left(1 + \frac{1}{6}\frac{ht_w}{b_ft_f}\right)}}$$

(b) When $M_e > M_y$

$$M_n = \left(1.92 - 1.17\sqrt{\frac{M_y}{M_e}}\right)M_y \le 1.5M_y \quad \text{(F10-3)}$$

where

M_e, the elastic *lateral-torsional buckling* moment, is determined as follows:

(i) For bending about the major principal axis of equal-leg angles:

$$M_e = \frac{0.46Eb^2t^2C_b}{L_b} \quad \text{(F10-4)}$$

(ii) For bending about the major principal axis of unequal-leg angles:

$$M_e = \frac{4.9EI_zC_b}{L_b^2}\left(\sqrt{\beta_w^2 + 0.052\left(\frac{L_bt}{r_z}\right)^2} + \beta_w\right) \quad \text{(F10-5)}$$

where

C_b is computed using Equation F1-1 with a maximum value of 1.5
L_b = laterally *unbraced length* of member, in. (mm)
I_z = minor principal axis moment of inertia, in.^{4} (mm^4)
r_z = radius of gyration about the minor principal axis, in. (mm)
t = thickness of angle leg, in. (mm)
β_w = section property for unequal leg angles, positive for short legs in compression and negative for long legs in compression. If the long leg is in compression anywhere along the unbraced length of the member, the negative value of β_w shall be used.

User Note: The equation for β_w and values for common angle sizes are listed in the Commentary.

(iii) For bending moment about one of the *geometric axes* of an equal-leg angle with no axial compression

(a) And with no lateral-torsional restraint:

(i) With maximum compression at the toe

$$M_e = \frac{0.66Eb^4tC_b}{L_b^2}\left(\sqrt{1 + 0.78\left(\frac{L_bt}{b^2}\right)^2} - 1\right) \quad \text{(F10-6a)}$$

(ii) With maximum tension at the toe

$$M_e = \frac{0.66Eb^4tC_b}{L_b^2}\left(\sqrt{1 + 0.78\left(\frac{L_bt}{b^2}\right)^2} + 1\right) \quad \text{(F10-6b)}$$

M_y shall be taken as 0.80 times the *yield moment* calculated using the geometric section modulus.

where

b = full width of leg in compression, in. (mm)

User Note: M_n may be taken as M_y for single angles with their vertical leg toe in compression, and having a span-to-depth ratio less than or equal to

$$\frac{1.64E}{F_y}\sqrt{\left(\frac{t}{b}\right)^2 - 1.4\frac{F_y}{E}}$$

(b) And with lateral-torsional restraint at the point of maximum moment only:

M_e shall be taken as 1.25 times M_e computed using Equation F10-6a or F10-6b.

M_y shall be taken as the yield moment calculated using the geometric section modulus.

3. Leg Local Buckling

The *limit state* of leg *local buckling* applies when the toe of the leg is in compression.

(a) For *compact sections,* the limit state of leg local buckling does not apply.
(b) For sections with noncompact legs:

$$M_n = F_yS_c\left(2.43 - 1.72\left(\frac{b}{t}\right)\sqrt{\frac{F_y}{E}}\right) \quad \text{(F10-7)}$$

(c) For sections with slender legs:

$$M_n = F_{cr}S_c \quad \text{(F10-8)}$$

where

$$F_{cr} = \frac{0.71E}{\left(\frac{b}{t}\right)^2} \quad \text{(F10-9)}$$

S_c = elastic section modulus to the toe in compression relative to the axis of bending, in.^{3} (mm^3). For bending about one of the *geometric axes* of an equal-leg angle with no lateral-torsional restraint, S_c shall be 0.80 of the geometric axis section modulus.

F11. RECTANGULAR BARS AND ROUNDS

This section applies to rectangular bars bent about either *geometric axis* and rounds.

The *nominal flexural strength*, M_n, shall be the lower value obtained according to the *limit states* of *yielding* (*plastic moment*) and *lateral-torsional buckling*.

1. Yielding

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

$$M_n = M_p = F_y Z \le 1.6M_y \quad \text{(F11-1)}$$

2. Lateral-Torsional Buckling

(a) For rectangular bars with $\frac{0.08E}{F_y} < \frac{L_b d}{t^2} \le \frac{1.9E}{F_y}$ bent about their major axis:

$$M_n = C_b \left[1.52 - 0.274\left(\frac{L_b d}{t^2}\right)\frac{F_y}{E}\right] M_y \le M_p \quad \text{(F11-2)}$$

(b) For rectangular bars with $\frac{L_b d}{t^2} > \frac{1.9E}{F_y}$ bent about their major axis:

$$M_n = F_{cr} S_x \le M_p \quad \text{(F11-3)}$$

where

$$F_{cr} = \frac{1.9EC_b}{\frac{L_b d}{t^2}} \quad \text{(F11-4)}$$

L_b = length between points that are either braced against lateral displacement of the compression region, or between points braced to prevent twist of the cross section, in. (mm)
d = depth of rectangular bar, in. (mm)
t = width of rectangular bar parallel to axis of bending, in. (mm)

(c) For rounds and rectangular bars bent about their minor axis, the *limit state* of *lateral-torsional buckling* need not be considered.

F12. UNSYMMETRICAL SHAPES

This section applies to all unsymmetrical shapes, except single angles.

The *nominal flexural strength*, M_n, shall be the lowest value obtained according to the *limit states* of *yielding (yield moment), lateral-torsional buckling,* and *local buckling* where

$$M_n = F_n S_{min} \quad \text{(F12-1)}$$

where
S_{min} = lowest elastic section modulus relative to the axis of bending, in.^{3} (mm^3)

1. Yielding

$$F_n = F_y \quad \text{(F12-2)}$$

2. Lateral-Torsional Buckling

$$F_n = F_{cr} \le F_y \quad \text{(F12-3)}$$

where
F_{cr} = *lateral-torsional buckling stress* for the section as determined by analysis, ksi (MPa)

User Note: In the case of Z-shaped members, it is recommended that F_{cr} be taken as $0.5F_{cr}$ of a channel with the same flange and web properties.

3. Local Buckling

$$F_n = F_{cr} \le F_y \quad \text{(F12-4)}$$

where
F_{cr} = *local buckling stress* for the section as determined by analysis, ksi (MPa)

F13. PROPORTIONS OF BEAMS AND GIRDERS

1. Strength Reductions for Members With Holes in the Tension Flange

This section applies to rolled or *built-up shapes* and cover-plated *beams* with holes, proportioned on the basis of flexural strength of the gross section.

In addition to the *limit states* specified in other sections of this Chapter, the *nominal flexural strength*, M_n, shall be limited according to the limit state of *tensile rupture* of the tension flange.

(a) When $F_u A_{fn} \ge Y_t F_y A_{fg}$, the limit state of tensile rupture does not apply.
(b) When $F_u A_{fn} < Y_t F_y A_{fg}$, the nominal flexural strength, M_n, at the location of the holes in the tension flange shall not be taken greater than

$$M_n = \frac{F_u A_{fn}}{A_{fg}} S_x \quad \text{(F13-1)}$$

where
A_{fg} = gross area of tension flange, calculated in accordance with the provisions of Section B4.3a, in.^{2} (mm^2)
A_{fn} = *net area* of tension flange, calculated in accordance with the provisions of Section B4.3b, in.^{2} (mm^2)
Y_t = 1.0 for $F_y/F_u \le 0.8$
= 1.1 otherwise

2. Proportioning Limits for I-Shaped Members

Singly symmetric I-shaped members shall satisfy the following limit:

$$0.1 \le \frac{I_{yc}}{I_y} \le 0.9 \quad \text{(F13-2)}$$

CHAPTER G

DESIGN OF MEMBERS FOR SHEAR

This chapter addresses webs of singly or doubly symmetric members subject to shear in the plane of the web, single angles and *HSS* sections, and shear in the weak direction of singly or doubly symmetric shapes.

The chapter is organized as follows:

User Note: For cases not included in this chapter, the following sections apply:
- H3.3 Unsymmetric sections
- J4.2 Shear strength of connecting elements
- J10.6 Web *panel zone* shear

G1. GENERAL PROVISIONS

Two methods of calculating shear strength are presented below. The method presented in Section G2 does not utilize the post *buckling strength* of the member (*tension field action*). The method presented in Section G3 utilizes tension field action.

The *design shear strength*, $\phi_v V_n$, and the *allowable shear strength*, V_n/Ω_v, shall be determined as follows:

For all provisions in this chapter except Section G2.1(a):

$$\phi_v = 0.90 \text{ (LRFD)} \qquad \Omega_v = 1.67 \text{ (ASD)}$$

G2. MEMBERS WITH UNSTIFFENED OR STIFFENED WEBS

1. Shear Strength

This section applies to webs of singly or doubly symmetric members and channels subject to shear in the plane of the web.

The *nominal shear strength*, V_n, of unstiffened or stiffened webs according to the *limit states* of *shear yielding* and *shear buckling*, is

$$V_n = 0.6F_y A_w C_v \qquad \text{(G2-1)}$$

(a) For webs of rolled I-shaped members with $h/t_w \le 2.24\sqrt{E/F_y}$:

$$\phi_v = 1.00 \text{ (LRFD)} \qquad \Omega_v = 1.50 \text{ (ASD)}$$

and

$$C_v = 1.0 \qquad \text{(G2-2)}$$

User Note: All current ASTM A6 W, S and HP shapes except W44×230, W40×149, W36×135, W33×118, W30×90, W24×55, W16×26 and W12×14 meet the criteria stated in Section G2.1(a) for F_y = 50 ksi (345 MPa).

(b) For webs of all other doubly symmetric shapes and singly symmetric shapes and channels, except round *HSS*, the web shear coefficient, C_v, is determined as follows:

(i) When $h/t_w \le 1.10\sqrt{k_v E/F_y}$

$$C_v = 1.0 \qquad \text{(G2-3)}$$

(ii) When $1.10\sqrt{k_v E/F_y} < h/t_w \le 1.37\sqrt{k_v E/F_y}$

$$C_v = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} \qquad \text{(G2-4)}$$

(iii) When $h/t_w > 1.37\sqrt{k_v E/F_y}$

$$C_v = \frac{1.51 k_v E}{(h/t_w)^2 F_y} \qquad \text{(G2-5)}$$

where

A_w = area of web, the overall depth times the web thickness, dt_w, in.^{2} (mm^2)
h = for rolled shapes, the clear distance between flanges less the fillet or corner radii, in. (mm)
= for built-up welded sections, the clear distance between flanges, in. (mm)
= for built-up bolted sections, the distance between *fastener* lines, in. (mm)
= for tees, the overall depth, in. (mm)
t_w = thickness of web, in. (mm)

The web plate *shear buckling* coefficient, k_v, is determined as follows:

(i) For webs without *transverse stiffeners* and with $h/t_w < 260$:

$$k_v = 5$$

except for the stem of tee shapes where $k_v = 1.2$.

CHAPTER H

DESIGN OF MEMBERS FOR COMBINED FORCES AND TORSION

This chapter addresses members subject to axial *force* and flexure about one or both axes, with or without torsion, and members subject to torsion only.

The chapter is organized as follows:

User Note: For *composite* members, see Chapter I.

H1. DOUBLY AND SINGLY SYMMETRIC MEMBERS SUBJECT TO FLEXURE AND AXIAL FORCE

1. Doubly and Singly Symmetric Members Subject to Flexure and Compression

The interaction of flexure and compression in doubly symmetric members and singly symmetric members for which $0.1 \le (I_{yc}/I_y) \le 0.9$, constrained to bend about a *geometric axis* (x and/or y) shall be limited by Equations H1-1a and H1-1b, where I_{yc} is the moment of inertia of the compression flange about the y-axis, in.^{4} (mm^4).

User Note: Section H2 is permitted to be used in lieu of the provisions of this section.

(a) When $\dfrac{P_r}{P_c} \ge 0.2$

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \le 1.0 \qquad \text{(H1-1a)}$$

(b) When $\dfrac{P_r}{P_c} < 0.2$

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \le 1.0 \qquad \text{(H1-1b)}$$

where

P_r = *required axial strength* using *LRFD* or *ASD load combinations,* kips (N)
P_c = *available axial strength*, kips (N)

M_r = *required flexural strength* using LRFD or ASD load combinations, kip-in. (N-mm)
M_c = *available flexural strength*, kip-in. (N-mm)
x = subscript relating symbol to *strong axis* bending
y = subscript relating symbol to *weak axis* bending

For design according to Section B3.3 (LRFD):

P_r = required axial strength using LRFD load combinations, kips (N)
P_c = $\phi_c P_n$ = *design axial strength*, determined in accordance with Chapter E, kips (N)
M_r = required flexural strength using LRFD load combinations, kip-in. (N-mm)
M_c = $\phi_b M_n$ = *design flexural strength* determined in accordance with Chapter F, kip-in. (N-mm)
ϕ_c = *resistance factor* for compression = 0.90
ϕ_b = resistance factor for flexure = 0.90

For design according to Section B3.4 (ASD):

P_r = required axial strength using ASD load combinations, kips (N)
P_c = P_n/Ω_c = *allowable axial strength*, determined in accordance with Chapter E, kips (N)
M_r = required flexural strength using ASD load combinations, kip-in. (N-mm)
M_c = M_n/Ω_b = *allowable flexural strength* determined in accordance with Chapter F, kip-in. (N-mm)
Ω_c = *safety factor* for compression = 1.67
Ω_b = safety factor for flexure = 1.67

2. Doubly and Singly Symmetric Members Subject to Flexure and Tension

The interaction of flexure and tension in doubly symmetric members and singly symmetric members constrained to bend about a *geometric axis* (x and/or y) shall be limited by Equations H1-1a and H1-1b

where

For design according to Section B3.3 (LRFD):

P_r = *required axial strength* using *LRFD load combinations*, kips (N)
P_c = $\phi_t P_n$ = *design axial strength*, determined in accordance with Section D2, kips (N)
M_r = required flexural strength using LRFD load combinations, kip-in. (N-mm)
M_c = $\phi_b M_n$ = design flexural strength determined in accordance with Chapter F, kip-in. (N-mm)
ϕ_t = *resistance factor* for tension (see Section D2)
ϕ_b = resistance factor for flexure = 0.90

For design according to Section B3.4 (ASD):

P_r = required axial strength using *ASD load combinations*, kips (N)
P_c = P_n/Ω_t = *allowable axial strength*, determined in accordance with Section D2, kips (N)