User Note: For all ASTM A6 W, S, M and HP shapes, when $F_y \leq 50$ ksi (345 MPa), $C_v = 1.0$.

G8. BEAMS AND GIRDERS WITH WEB OPENINGS

The effect of all web openings on the nominal shear strength of steel and *composite beams* shall be determined. Adequate reinforcement shall be provided when the *required strength* exceeds the *available strength* of the member at the opening.

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 to 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 in Flexure and Compression

The interaction of flexure and compression in doubly symmetric members and singly symmetric members for which $0.1 \leq (I_{yc}/I_y) \leq 0.9$, that are 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 about the y-axis referred to the compression flange, in.^{4} (mm^4).

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

(a) For $\dfrac{P_r}{P_c} \geq 0.2$

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

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

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

where

P_r = required axial compressive strength, kips (N)
P_c = *available axial compressive strength*, kips (N)
M_r = *required flexural strength*, 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 compressive strength* using *LRFD load combinations*, kips (N)
$P_c = \phi_c P_n$ = *design axial compressive 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 compressive strength using *ASD load combinations*, kips (N)
$P_c = P_n/\Omega_c$ = *allowable axial compressive strength*, determined in accordance with Chapter E, kips (N)
M_r = required flexural strength using ASD load combinations, kip-in. (N-mm)
$M_c = M_n/\Omega_b$ = *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 in 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 tensile strength* using LRFD *load combinations*, kips (N)
$P_c = \phi_t P_n$ = *design tensile 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 doubly symmetric members, C_b in Chapter F may be increased by $\sqrt{1+\frac{P_u}{P_{ey}}}$ for axial tension that acts concurrently with flexure,

where

$$P_{ey} = \frac{\pi^2 EI_y}{L_b^2}$$

For design according to Section B3.4 (ASD)

P_r = required tensile strength using *ASD load combinations*, kips (N)
P_c = P_n/Ω_t = *allowable tensile strength*, determined in accordance with Section D2, 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_t = *safety factor* for tension (see Section D2)
Ω_b = safety factor for flexure = 1.67

For doubly symmetric members, C_b in Chapter F may be increased by $\sqrt{1+\frac{1.5P_a}{P_{ey}}}$ for axial tension that acts concurrently with flexure

where

$$P_{ey} = \frac{\pi^2 EI_y}{L_b^2}$$

A more detailed analysis of the interaction of flexure and tension is permitted in lieu of Equations H1-1a and H1-1b.

3. Doubly Symmetric Members in Single Axis Flexure and Compression

For doubly symmetric members in flexure and compression with moments primarily in one plane, it is permissible to consider the two independent *limit states*, *in-plane instability* and *out-of-plane buckling* or *flexural-torsional buckling*, separately in lieu of the combined approach provided in Section H1.1.

(a) For the limit state of in-plane instability, Equations H1-1 shall be used with P_c, M_r, and M_c determined in the plane of bending.
(b) For the limit state of out-of-plane buckling

$$\frac{P_r}{P_{co}} + \left(\frac{M_r}{M_{cx}}\right)^2 \le 1.0 \qquad \text{(H1-2)}$$

where

P_{co} = *available compressive strength* out of the plane of bending, kips (N)
M_{cx} = *available flexural-torsional strength* for *strong axis* flexure determined from Chapter F, kip-in. (N-mm)

If bending occurs only about the *weak axis*, the moment ratio in Equation H1-2 shall be neglected.

For members with significant biaxial moments ($M_r/M_c \geq 0.05$ in both directions), the provisions of Section H1.1 shall be followed.

H2. UNSYMMETRIC AND OTHER MEMBERS SUBJECT TO FLEXURE AND AXIAL FORCE

This section addresses the interaction of flexure and axial *stress* for shapes not covered in Section H1. It is permitted to use the provisions of this Section for any shape in lieu of the provisions of Section H1.

$$\left| \frac{f_a}{F_a} + \frac{f_{bw}}{F_{bw}} + \frac{f_{bz}}{F_{bz}} \right| \leq 1.0 \qquad \text{(H2-1)}$$

where

f_a = required axial stress at the point of consideration, ksi (MPa)
F_a = *available axial stress* at the point of consideration, ksi (MPa)
f_{bw}, f_{bz} = required flexural stress at the point of consideration, ksi (MPa)
F_{bw}, F_{bz} = *available flexural stress* at the point of consideration, ksi (MPa)
w = subscript relating symbol to major principal axis bending
z = subscript relating symbol to minor principal axis bending

For design according to Section B3.3 (LRFD)

f_a = required axial stress using *LRFD load combinations*, ksi (MPa)
F_a = $\phi_c F_{cr}$ = *design axial stress*, determined in accordance with Chapter E for compression or Section D2 for tension, ksi (MPa)
f_{bw}, f_{bz} = required flexural *stress* at the specific location in the cross section using LRFD load combinations, ksi (MPa)
F_{bw}, F_{bz} = $\dfrac{\phi_b M_n}{S}$ = *design flexural stress* determined in accordance with Chapter F, ksi (MPa). Use the section modulus for the specific location in the cross section and consider the sign of the stress.
ϕ_c = *resistance factor* for compression = 0.90
ϕ_t = resistance factor for tension (Section D2)
ϕ_b = resistance factor for flexure = 0.90

For design according to Section B3.4 (ASD)

f_a = required axial stress using ASD *load combinations*, ksi (MPa)
F_a = $\dfrac{F_{cr}}{\Omega_c}$ = *allowable axial stress* determined in accordance with Chapter E for compression, or Section D2 for tension, ksi (MPa)
f_{bw}, f_{bz} = required flexural stress at the specific location in the cross section using *ASD load combinations*, ksi (MPa)

$F_{bw}, F_{bz} = \dfrac{M_n}{\Omega_b S}$ = *allowable flexural stress* determined in accordance with Chapter F, ksi (MPa). Use the section modulus for the specific location in the cross section and consider the sign of the stress.

Ω_c = *safety factor* for compression = 1.67

Ω_t = safety factor for tension (Section D2)

Ω_b = safety factor for flexure = 1.67

Equation H2-1 shall be evaluated using the principal bending axes by considering the sense of the flexural stresses at the critical points of the cross section. The flexural terms are either added to or subtracted from the axial term as appropriate. When the axial *force* is compression, second order effects shall be included according to the provisions of Chapter C.

A more detailed analysis of the interaction of flexure and tension is permitted in lieu of Equation H2-1.

H3. MEMBERS UNDER TORSION AND COMBINED TORSION, FLEXURE, SHEAR AND/OR AXIAL FORCE

1. Torsional Strength of Round and Rectangular HSS

The *design torsional strength*, $\phi_T T_n$, and the *allowable torsional strength*, T_n/Ω_T, for round and rectangular *HSS* shall be determined as follows:

$$\phi_T = 0.90 \text{ (LRFD)} \qquad \Omega_T = 1.67 \text{ (ASD)}$$

The nominal torsional strength, T_n, according to the *limit states* of *torsional yielding* and *torsional buckling* is:

$$T_n = F_{cr} C \tag{H3-1}$$

where

C is the *HSS* torsional constant

F_{cr} shall be determined as follows:

(a) For round *HSS*, F_{cr} shall be the larger of

$$F_{cr} = \frac{1.23E}{\sqrt{\dfrac{L}{D}}\left(\dfrac{D}{t}\right)^{\frac{5}{4}}} \tag{H3-2a}$$

and

$$F_{cr} = \frac{0.60E}{\left(\dfrac{D}{t}\right)^{\frac{3}{2}}} \tag{H3-2b}$$

but shall not exceed $0.6F_y$,

where

L = length of the member, in. (mm)

D = outside diameter, in. (mm)

(b) For rectangular *HSS*

(i) For $h/t \le 2.45\sqrt{E/F_y}$

$$F_{cr} = 0.6F_y \tag{H3-3}$$

(ii) For $2.45\sqrt{E/F_y} < h/t \le 3.07\sqrt{E/F_y}$

$$F_{cr} = 0.6F_y(2.45\sqrt{E/F_y})/(h/t) \tag{H3-4}$$

(iii) For $3.07\sqrt{E/F_y} < h/t \le 260$

$$F_{cr} = 0.458\pi^2 E/(h/t)^2 \tag{H3-5}$$

User Note: The torsional shear constant, C, may be conservatively taken as:

For a round *HSS*: $C = \dfrac{\pi(D-t)^2 t}{2}$

For rectangular *HSS*: $C = 2(B-t)(H-t)t - 4.5(4-\pi)t^3$

2. HSS Subject to Combined Torsion, Shear, Flexure and Axial Force

When the *required torsional strength*, T_r, is less than or equal to 20 percent of the *available torsional strength*, T_c, the interaction of torsion, shear, flexure and/or axial *force* for *HSS* shall be determined by Section H1 and the torsional effects shall be neglected. When T_r exceeds 20 percent of T_c, the interaction of torsion, shear, flexure and/or axial force shall be limited by

$$\left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right) + \left(\frac{V_r}{V_c} + \frac{T_r}{T_c}\right)^2 \le 1.0 \tag{H3-6}$$

where

For design according to Section B3.3 (LRFD)

P_r = *required axial strength* using *LRFD load combinations*, kips (N)
P_c = ϕP_n, *design tensile* or *compressive strength* in accordance with Chapter D or 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* in accordance with Chapter F, kip-in. (N-mm)
V_r = *required shear strength* using LRFD load combinations, kips (N)
V_c = $\phi_v V_n$, *design shear strength* in accordance with Chapter G, kips (N)
T_r = *required torsional strength* using LRFD load combinations, kip-in. (N-mm)
T_c = $\phi_T T_n$, *design torsional strength* in accordance with Section H3.1, kip-in. (N-mm)

For design according to Section B3.4 (ASD)

P_r = required axial strength using *ASD load combinations*, kips (N)
P_c = P_n/Ω, *allowable tensile* or *compressive strength* in accordance with Chapter D or E, kips (N)
M_r = required flexural strength using ASD load combinations determined in accordance with Section B5, kip-in. (N-mm)
M_c = M_n/Ω_b, *allowable flexural strength* in accordance with Chapter F, kip-in. (N-mm)
V_r = required shear strength using ASD load combinations, kips (N)
V_c = V_n/Ω_v, *allowable shear strength* in accordance with Chapter G, kips (N)
T_r = required torsional strength using ASD load combinations, kip-in. (N-mm)
T_c = T_n/Ω_T, *allowable torsional strength* in accordance with Section H3.1, kip-in. (N-mm)

3. Strength of Non-HSS Members under Torsion and Combined Stress

The *design torsional strength*, $\phi_T F_n$, and the *allowable torsional strength*, F_n/Ω_T, for non-*HSS* members shall be the lowest value obtained according to the *limit states* of *yielding* under normal *stress*, *shear yielding* under shear stress, or *buckling*, determined as follows:

$$\phi_T = 0.90 \text{ (LRFD)} \qquad \Omega_T = 1.67 \text{ (ASD)}$$

(a) For the limit state of yielding under normal stress

$$F_n = F_y \tag{H3-7}$$

(b) For the limit state of shear yielding under shear stress

$$F_n = 0.6F_y \tag{H3-8}$$

(c) For the limit state of buckling

$$F_n = F_{cr} \tag{H3-9}$$

where

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

Some constrained local yielding is permitted adjacent to areas that remain elastic.

CHAPTER I

DESIGN OF COMPOSITE MEMBERS

This chapter addresses *composite columns* composed of rolled or built-up structural steel shapes or *HSS*, and structural concrete acting together, and steel *beams* supporting a reinforced concrete slab so interconnected that the beams and the slab act together to resist bending. Simple and continuous *composite beams* with *shear connectors* and *concrete-encased beams*, constructed with or without temporary shores, are included.

The chapter is organized as follows:

I1. GENERAL PROVISIONS

In determining *load effects* in members and *connections* of a structure that includes *composite* members, consideration shall be given to the effective sections at the time each increment of *load* is applied. The design, detailing and material properties related to the concrete and reinforcing steel portions of composite construction shall comply with the reinforced concrete and reinforcing bar design specifications stipulated by the *applicable building code*. In the absence of a building code, the provisions in ACI 318 shall apply.

1. Nominal Strength of Composite Sections

Two methods are provided for determining the *nominal strength* of *composite* sections: the *plastic stress distribution method* and the *strain-compatibility method.*

The *tensile strength* of the concrete shall be neglected in the determination of the nominal strength of composite members.

1a. Plastic Stress Distribution Method

For the plastic stress distribution method, the nominal strength shall be computed assuming that steel components have reached a *stress* of F_y in either tension or compression and concrete components in compression have reached a *stress* of $0.85f'_c$. For round *HSS* filled with concrete, a *stress* of $0.95f'_c$ is permitted to be used for concrete components in uniform compression to account for the effects of concrete confinement.

1b. Strain-Compatibility Method

For the strain compatibility method, a linear distribution of strains across the section shall be assumed, with the maximum concrete compressive strain equal to 0.003 in./in. (mm/mm). The stress-strain relationships for steel and concrete shall be obtained from tests or from published results for similar materials.

> **User Note**: The strain compatibility method should be used to determine nominal strength for irregular sections and for cases where the steel does not exhibit elasto-plastic behavior. General guidelines for the strain-compatibility method for encased columns are given in AISC Design Guide 6 and ACI 318 Sections 10.2 and 10.3.

2. Material Limitations

Concrete and steel reinforcing bars in *composite* systems shall be subject to the following limitations.

(1) For the determination of the *available strength*, concrete shall have a compressive strength f_c' of not less than 3 ksi (21 MPa) nor more than 10 ksi (70 MPa) for normal weight concrete and not less than 3 ksi (21 MPa) nor more than 6 ksi (42 MPa) for lightweight concrete.

> **User Note:** Higher strength concrete materials may be used for *stiffness* calculations but may not be relied upon for strength calculations unless justified by testing or analysis.

(2) The *specified minimum yield stress* of structural steel and reinforcing bars used in calculating the strength of a *composite column* shall not exceed 75 ksi (525 MPa).

Higher material strengths are permitted when their use is justified by testing or analysis.

> **User Note:** Additional reinforced concrete material limitations are specified in ACI 318.

3. Shear Connectors

Shear connectors shall be headed steel studs not less than four stud diameters in length after installation, or hot-rolled steel channels. Shear stud design values shall be taken as per Sections I2.1g and I3.2d(2). Stud connectors shall conform to the requirements of Section A3.6. Channel connectors shall conform to the requirements of Section A3.1.

I2. AXIAL MEMBERS

This section applies to two types of *composite* axial members: encased and filled sections.

1. Encased Composite Columns

1a. Limitations

To qualify as an encased *composite column*, the following limitations shall be met:

(1) The cross-sectional area of the steel core shall comprise at least 1 percent of the total composite cross section.

(2) Concrete encasement of the steel core shall be reinforced with continuous longitudinal bars and lateral ties or spirals. The minimum *transverse reinforcement* shall be at least 0.009 in.^{2} per in. (6 mm^2 per mm) of tie spacing.

(3) The minimum reinforcement ratio for continuous longitudinal reinforcing, ρ_{sr}, shall be 0.004, where ρ_{sr} is given by:

$$\rho_{sr} = \frac{A_{sr}}{A_g} \tag{I2-1}$$

where

A_{sr} = area of continuous reinforcing bars, in.^{2} (mm^2)
A_g = gross area of composite member, in.^{2} (mm^2)

1b. Compressive Strength

The *design compressive strength*, $\phi_c P_n$, and *allowable compressive strength*, P_n/Ω_c, for axially loaded *encased composite columns* shall be determined for the limit state of *flexural buckling* based on column slenderness as follows:

$$\phi_c = 0.75\ \text{(LRFD)} \qquad \Omega_c = 2.00\ \text{(ASD)}$$

(a) When $P_e \geq 0.44 P_o$

$$P_n = P_o\left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] \tag{I2-2}$$

(b) When $P_e < 0.44 P_o$

$$P_n = 0.877 P_e \tag{I2-3}$$

where

$$P_o = A_s F_y + A_{sr} F_{yr} + 0.85 A_c f_c' \tag{I2-4}$$

$$P_e = \pi^2 (EI_{eff})/(KL)^2 \tag{I2-5}$$

and where

A_s = area of the steel section, in.^{2} (mm^2)
A_c = area of concrete, in.^{2} (mm^2)
A_{sr} = area of continuous reinforcing bars, in.^{2} (mm^2)
E_c = modulus of elasticity of concrete = $w_c^{1.5}\sqrt{f_c'}$, ksi ($0.043 w_c^{1.5}\sqrt{f_c'}$, MPa)
E_s = modulus of elasticity of steel = 29,000 ksi (210 MPa)
f_c' = specified compressive strength of concrete, ksi (MPa)
F_y = *specified minimum yield stress* of steel section, ksi (MPa)
F_{yr} = specified minimum yield stress of reinforcing bars, ksi (MPa)
I_c = moment of inertia of the concrete section, in.^{4} (mm^4)
I_s = moment of inertia of steel shape, in.^{4} (mm^4)
I_{sr} = moment of inertia of reinforcing bars, in.^{4} (mm^4)

K = the effective length factor determined in accordance with Chapter C
L = laterally unbraced length of the member, in. (mm)
w_c = weight of concrete per unit volume ($90 \leq w_c \leq 155$ lbs/ft^3 or $1500 \leq w_c \leq 2500$ kg/m^3)

where

EI_{eff} = effective stiffness of composite section, kip-in.^{2} (N-mm^2)

$$EI_{eff} = E_s I_s + 0.5 E_s I_{sr} + C_1 E_c I_c \tag{I2-6}$$

where

$$C_1 = 0.1 + 2\left(\frac{A_s}{A_c + A_s}\right) \leq 0.3 \tag{I2-7}$$

1c. Tensile Strength

The *design tensile strength*, $\phi_t P_n$, and *allowable tensile strength*, P_n/Ω_t, for encased composite columns shall be determined for the limit state of *yielding* as

$$P_n = A_s F_y + A_{sr} F_{yr} \tag{I2-8}$$

$$\phi_t = 0.90\,(\text{LRFD}) \qquad \Omega_t = 1.67\,(\text{ASD})$$

1d. Shear Strength

The *available shear strength* shall be calculated based on either the shear strength of the steel section alone as specified in Chapter G plus the shear strength provided by tie reinforcement, if present, or the shear strength of the reinforced concrete portion alone.

User Note: The nominal shear strength of tie reinforcement may be determined as $A_{st} F_{yr}(d/s)$ where A_{st} is the area of tie reinforcement, d is the effective depth of the concrete section, and s is the spacing of the tie reinforcement. The shear capacity of reinforced concrete may be determined according to ACI 318, Chapter 11.

1e. Load Transfer

*Load*s applied to axially loaded encased composite columns shall be transferred between the steel and concrete in accordance with the following requirements:

(a) When the external *force* is applied directly to the steel section, *shear connectors* shall be provided to transfer the required shear force, V', as follows:

$$V' = V(1 - A_s F_y / P_o) \tag{I2-9}$$

where

V = required shear force introduced to *column*, kips (N)
A_s = area of steel cross section, in.^{2} (mm^2)
P_o = nominal axial compressive strength without consideration of *length effects*, kips (N)

(b) When the external force is applied directly to the concrete encasement, shear connectors shall be provided to transfer the required shear force, V', as follows:

$$V' = V(A_s F_y / P_o) \tag{I2-10}$$

(c) When load is applied to the concrete of an encased composite column by direct bearing the *design bearing strength*, $\phi_B P_p$, and the *allowable bearing strength*, P_p/Ω_B, of the concrete shall be:

$$P_p = 1.7 f_c' A_B \tag{I2-11}$$

$$\phi_B = 0.65 \text{ (LRFD)} \qquad \Omega_B = 2.31 \text{ (ASD)}$$

where

A_B = loaded area of concrete, in.^{2} (mm^2)

1f. Detailing Requirements

At least four continuous longitudinal reinforcing bars shall be used in encased composite columns. *Transverse reinforcement* shall be spaced at the smallest of 16 longitudinal bar diameters, 48 tie bar diameters or 0.5 times the least dimension of the composite section. The encasement shall provide at least 1.5 in. (38 mm) of clear cover to the reinforcing steel.

Shear connectors shall be provided to transfer the required shear *force* specified in Section I2.1e. The shear connectors shall be distributed along the length of the member at least a distance of 2.5 times the depth of the encased composite column above and below the load transfer region. The maximum connector spacing shall be 16 in. (405 mm). Connectors to transfer axial load shall be placed on at least two faces of the steel shape in a configuration symmetrical about the steel shape axes.

If the composite cross section is built up from two or more encased steel shapes, the shapes shall be interconnected with *lacing*, *tie plates*, *batten plates* or similar components to prevent buckling of individual shapes due to loads applied prior to hardening of the concrete.

1g. Strength of Stud Shear Connectors

The *nominal strength* of one stud shear connector embedded in solid concrete is:

$$Q_n = 0.5 A_{sc} \sqrt{f_c' E_c} \le A_{sc} F_u \tag{I2-12}$$

where

A_{sc} = cross-sectional area of stud shear connector, in.^{2} (mm^2)
F_u = *specified minimum tensile strength* of a stud shear connector, ksi (MPa)

2. Filled Composite Columns

2a. Limitations

To qualify as a filled *composite column* the following limitations shall be met:

(1) The cross-sectional area of the steel *HSS* shall comprise at least 1 percent of the total *composite* cross section.
(2) The maximum b/t ratio for a rectangular HSS used as a composite column shall be equal to $2.26\sqrt{E/F_y}$. Higher ratios are permitted when their use is justified by testing or analysis.

(3) The maximum D/t ratio for a round HSS filled with concrete shall be $0.15\ E/F_y$. Higher ratios are permitted when their use is justified by testing or analysis.

2b. Compressive Strength

The *design compressive strength*, $\phi_c P_n$, and *allowable compressive strength*, P_n/Ω_c, for axially loaded filled composite columns shall be determined for the *limit state* of *flexural buckling* based on Section I2.1b with the following modifications:

$$P_o = A_s F_y + A_{sr} F_{yr} + C_2 A_c f'_c \tag{I2-13}$$

$C_2 = 0.85$ for rectangular sections and 0.95 for circular sections

$$EI_{eff} = E_s I_s + E_s I_{sr} + C_3 E_c I_c \tag{I2-14}$$

$$C_3 = 0.6 + 2\left(\frac{A_s}{A_c + A_s}\right) \leq 0.9 \tag{I2-15}$$

2c. Tensile Strength

The *design tensile strength*, $\phi_t P_n$, and *allowable tensile strength*, P_n/Ω_t, for filled composite columns shall be determined for the limit state of *yielding* as:

$$P_n = A_s F_y + A_{sr} F_{yr} \tag{I2-16}$$

$$\phi_t = 0.90\,(\text{LRFD}) \qquad \Omega_t = 1.67\,(\text{ASD})$$

2d. Shear Strength

The *available shear strength* shall be calculated based on either the shear strength of the steel section alone as specified in Chapter G or the shear strength of the reinforced concrete portion alone.

User Note: The shear strength of reinforced concrete may be determined by ACI 318, Chapter 11.

2e. Load Transfer

Loads applied to filled composite columns shall be transferred between the steel and concrete. When the external force is applied either to the steel section or to the concrete infill, transfer of force from the steel section to the concrete core is required from *direct bond interaction*, shear *connection* or direct bearing. The force transfer *mechanism* providing the largest *nominal strength* may be used. These force transfer mechanisms shall not be superimposed.

When load is applied to the concrete of an encased or filled composite column by direct bearing the *design bearing strength*, $\phi_B P_p$, and the *allowable bearing strength*, P_p/Ω_B, of the concrete shall be:

$$P_p = 1.7 f'_c A_B \tag{I2-17}$$

$$\phi_B = 0.65\,(\text{LRFD}) \qquad \Omega_B = 2.31\,(\text{ASD})$$

where

A_B is the loaded area, in.^{2} (mm^2)

2f. Detailing Requirements

Where required, shear connectors transferring the required shear force shall be distributed along the length of the member at least a distance of 2.5 times the width of a rectangular HSS or 2.5 times the diameter of a round HSS both above and below the load transfer region. The maximum connector spacing shall be 16 in. (405 mm).

I3. FLEXURAL MEMBERS

1. General

1a. Effective Width

The *effective width* of the concrete slab is the sum of the effective widths for each side of the *beam* centerline, each of which shall not exceed:

(1) one-eighth of the beam span, center-to-center of supports;
(2) one-half the distance to the centerline of the adjacent beam; or
(3) the distance to the edge of the slab.

1b. Shear Strength

The available shear strength of *composite beams* with *shear connectors* shall be determined based upon the properties of the steel section alone in accordance with Chapter G. The available shear strength of concrete-encased and filled *composite* members shall be determined based upon the properties of the steel section alone in accordance with Chapter G or based upon the properties of the concrete and longitudinal steel reinforcement.

> **User Note**: The shear strength of the reinforced concrete may be determined in accordance with ACI 318, Chapter 11.

1c. Strength During Construction

When temporary shores are not used during construction, the steel section alone shall have adequate strength to support all *load*s applied prior to the concrete attaining 75 percent of its specified strength f'_c. The available flexural strength of the steel section shall be determined according to Chapter F.

2. Strength of Composite Beams with Shear Connectors

2a. Positive Flexural Strength

The *design positive flexural strength*, $\phi_b M_n$, and the *allowable positive flexural strength*, M_n/Ω_b, shall be determined for the *limit state* of *yielding* as follows:

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

(a) For $h/t_w \leq 3.76\sqrt{E/F_y}$,

M_n shall be determined from the plastic *stress* distribution on the *composite* section for the limit state of *yielding* (*plastic moment*).

User Note: All current ASTM A6 W, S and HP shapes satisfy the limit given in Section I3.2a(a) for $F_y \leq 50$ ksi (345 MPa).

(b) For $h/t_w > 3.76\sqrt{E/F_y}$,

M_n shall be determined from the superposition of elastic stresses, considering the effects of shoring, for the limit state of *yielding* (*yield moment*).

2b. Negative Flexural Strength

The *design negative flexural strength*, $\phi_b M_n$, and *the allowable negative flexural strength*, M_n/Ω_b, shall be determined for the steel section alone, in accordance with the requirements of Chapter F.

Alternatively, the available negative flexural strength shall be determined from the plastic stress distribution on the *composite* section, for the *limit state* of *yielding (plastic moment),* with

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

provided that:

(1) The steel *beam* is *compact* and is adequately braced according to Chapter F.
(2) *Shear connectors* connect the slab to the steel beam in the negative moment region.
(3) The slab reinforcement parallel to the steel beam, within the *effective width* of the slab, is *properly developed*.

2c. Strength of Composite Beams with Formed Steel Deck

(1) General

The *available flexural strength* of composite construction consisting of concrete slabs on *formed steel deck* connected to steel beams shall be determined by the applicable portions of Section I3.2a and I3.2b, with the following requirements:

(a) This section is applicable to decks with *nominal rib height* not greater than 3 in. (75 mm). The average width of concrete rib or haunch, w_r, shall be not less than 2 in. (50 mm), but shall not be taken in calculations as more than the minimum clear width near the top of the steel deck.

(b) The concrete slab shall be connected to the steel beam with welded stud shear connectors $^3/_4$ in. (19 mm) or less in diameter (AWS D1.1). Studs shall be welded either through the deck or directly to the steel cross section. Stud shear connectors, after installation, shall extend not less than $1^1/_2$ in. (38 mm) above the top of the steel deck and there shall be at least $^1/_2$ in. (13 mm) of concrete cover above the top of the installed studs.

(c) The slab thickness above the steel deck shall be not less than 2 in. (50 mm).

(d) Steel deck shall be anchored to all supporting members at a spacing not to exceed 18 in. (460 mm). Such anchorage shall be provided by stud connectors, a combination of stud connectors and arc spot (puddle) welds, or other devices specified by the designer.

(2) Deck Ribs Oriented Perpendicular to Steel Beam

Concrete below the top of the steel deck shall be neglected in determining composite section properties and in calculating A_c for deck ribs oriented perpendicular to the steel beams.

(3) Deck Ribs Oriented Parallel to Steel Beam

Concrete below the top of the steel deck may be included in determining composite section properties and shall be included in calculating A_c.

Formed steel deck ribs over supporting beams may be split longitudinally and separated to form a *concrete haunch*.

When the nominal depth of steel deck is $1^1/_2$ in. (38 mm) or greater, the average width, w_r, of the supported haunch or rib shall be not less than 2 in. (50 mm) for the first stud in the transverse row plus four stud diameters for each additional stud.

2d. Shear Connectors

(1) Load Transfer for Positive Moment

The entire *horizontal shear* at the interface between the steel *beam* and the concrete slab shall be assumed to be transferred by shear connectors, except for *concrete-encased beams* as defined in Section I3.3. For *composite* action with concrete subject to flexural compression, the total horizontal shear force, V', between the point of maximum positive moment and the point of zero moment shall be taken as the lowest value according to the *limit states* of *concrete crushing*, *tensile yielding* of the steel section, or strength of the shear connectors:

(a) Concrete crushing

$$V' = 0.85 f_c' A_c \tag{I3-1a}$$

(b) Tensile yielding of the steel section

$$V' = F_y A_s \tag{I3-1b}$$

(c) Strength of shear connectors

$$V' = \Sigma Q_n \tag{I3-1c}$$

where

A_c = area of concrete slab within *effective width*, in.^{2} (mm^2)

A_s = area of steel cross section, in.^{2} (mm^2)

ΣQ_n = sum of *nominal strengths* of shear connectors between the point of maximum positive moment and the point of zero moment, kips (N)

(2) Load Transfer for Negative Moment

In continuous *composite beams* where longitudinal reinforcing steel in the negative moment regions is considered to act compositely with the steel *beam*, the total *horizontal shear force* between the point of maximum negative moment and the point of zero moment shall be taken as the lower value according to the limit states of *yielding* of the steel reinforcement in the slab, or strength of the shear connectors:

(a) Tensile yielding of the slab reinforcement

$$V' = A_r F_{yr} \tag{I3-2a}$$

where

A_r = area of adequately developed longitudinal reinforcing steel within the *effective width* of the concrete slab, in.^{2}(mm^2)

F_{yr} = *specified minimum yield stress* of the reinforcing steel, ksi (MPa)

(b) Strength of shear connectors

$$V' = \Sigma Q_n \tag{I3-2b}$$

(3) Strength of Stud Shear Connectors

The *nominal strength* of one stud shear connector embedded in solid concrete or in a *composite* slab is

$$Q_n = 0.5A_{sc}\sqrt{f_c' E_c} \leq R_g R_p A_{sc} F_u \tag{I3-3}$$

where

A_{sc} = cross-sectional area of stud *shear connector*, in.^{2} (mm^2)

E_c = modulus of elasticity of concrete $= w_c^{1.5}\sqrt{f_c'}$, ksi ($0.043w_c^{1.5}\sqrt{f_c'}$, MPa)

F_u = *specified minimum tensile strength* of a stud shear connector, ksi (MPa)

R_g = 1.0; (a) for one stud welded in a steel deck rib with the deck oriented perpendicular to the steel shape; (b) for any number of studs welded in a row directly to the steel shape; (c) for any number of studs welded in a row through steel deck with the deck oriented parallel to the steel shape and the ratio of the *average rib width* to rib depth ≥ 1.5

= 0.85; (a) for two studs welded in a steel deck rib with the deck oriented perpendicular to the steel shape; (b) for one stud welded through steel deck with the deck oriented parallel to the steel shape and the ratio of the average rib width to rib depth <1.5

= 0.7 for three or more studs welded in a steel deck rib with the deck oriented perpendicular to the steel shape

R_p = 1.0 for studs welded directly to the steel shape (in other words, not through steel deck or sheet) and having a haunch detail with not more than 50 percent of the top flange covered by deck or sheet steel closures

= 0.75; (a) for studs welded in a *composite* slab with the deck oriented perpendicular to the *beam* and $e_{mid\text{-}ht} \geq 2$ in. (50 mm); (b) for studs welded through steel deck, or steel sheet used as *girder filler* material, and embedded in a *composite* slab with the deck oriented parallel to the *beam*

= 0.6 for studs welded in a composite slab with deck oriented perpendicular to the beam and $e_{mid\text{-}ht} < 2$ in. (50 mm)

$e_{mid\text{-}ht}$ = distance from the edge of stud shank to the steel deck web, measured at mid-height of the deck rib, and in the *load* bearing direction of the stud (in other words, in the direction of maximum moment for a simply supported beam), in. (mm)

w_c = weight of concrete per unit volume ($90 \leq w_c \leq 155$ lbs/ft^3 or $1500 \leq w_c \leq 2500$ kg/m^3)

User Note: The table below presents values for R_g and R_p for several cases.

Condition	R_g	R_p
No decking*	1.0	1.0
Decking oriented parallel to the steel shape		
$\frac{w_r}{h_r} \geq 1.5$	1.0	0.75
$\frac{w_r}{h_r} < 1.5$	0.85**	0.75
Decking oriented perpendicular to the steel shape Number of studs occupying the same decking rib		
1	1.0	0.6+
2	0.85	0.6+
3 or more	0.7	0.6+

h_r = nominal rib height, in. (mm)

w_r = average width of concrete rib or haunch (as defined in Section I3.2c), in. (mm)

* to qualify as "no decking," stud shear connectors shall be welded directly to the steel shape and no more than 50 percent of the top flange of the steel shape may be covered by decking or sheet steel, such as girder filler material.

** for a single stud

+ this value may be increased to 0.75 when $e_{mid\text{-}ht} \geq 2$ in. (51 mm)

(4) Strength of Channel Shear Connectors

The nominal strength of one channel shear connector embedded in a solid concrete slab is

$$Q_n = 0.3(t_f + 0.5t_w)L_c\sqrt{f'_c E_c} \tag{I3-4}$$

where

t_f = flange thickness of channel shear connector, in. (mm)
t_w = web thickness of channel shear connector, in. (mm)
L_c = length of channel shear connector, in. (mm)

The strength of the channel shear connector shall be developed by welding the channel to the beam flange for a force equal to Q_n, considering eccentricity on the connector.

(5) Required Number of Shear Connectors

The number of shear connectors required between the section of maximum bending moment, positive or negative, and the adjacent section of zero moment shall be equal to the *horizontal shear force* as determined in Sections I3.2d(1) and I3.2d(2) divided by the nominal strength of one shear connector as determined from Section I3.2d(3) or Section I3.2d(4).

(6) Shear Connector Placement and Spacing

Shear connectors required on each side of the point of maximum bending moment, positive or negative, shall be distributed uniformly between that point and the adjacent points of zero moment, unless otherwise specified. However, the number of shear connectors placed between any concentrated *load* and the nearest point of zero moment shall be sufficient to develop the maximum moment required at the *concentrated load* point.

Shear connectors shall have at least 1 in. (25 mm) of lateral concrete cover, except for connectors installed in the ribs of *formed steel deck*s. The diameter of studs shall not be greater than 2.5 times the thickness of the flange to which they are welded, unless located over the web. The minimum center-to-center spacing of stud connectors shall be six diameters along the longitudinal axis of the supporting *composite beam* and four diameters transverse to the longitudinal axis of the supporting composite beam, except that within the ribs of formed steel decks oriented perpendicular to the steel beam the minimum center-to-center spacing shall be four diameters in any direction. The maximum center-to-center spacing of shear connectors shall not exceed eight times the total slab thickness nor 36 in.

3. Flexural Strength of Concrete-Encased and Filled Members

The *nominal flexural strength* of concrete-encased and filled members shall be determined using one of the following methods:

(a) The superposition of elastic *stresses* on the *composite* section, considering the effects of shoring, for the *limit state* of *yielding* (*yield moment*), where

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

(b) The plastic stress distribution on the steel section alone, for *the limit state* of *yielding (plastic moment)*, where

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

(c) If *shear connectors* are provided and the concrete meets the requirements of Section I1.2, the nominal flexural strength shall be computed based upon the plastic stress distribution on the composite section or from the strain-compatibility method, where

$$\phi_b = 0.85\,(\text{LRFD}) \qquad \Omega_b = 1.76\,(\text{ASD})$$

I4. COMBINED AXIAL FORCE AND FLEXURE

The interaction between axial forces and flexure in composite members shall account for stability as required by Chapter C. The *design compressive strength*, $\phi_c P_n$, and *allowable compressive strength*, P_n/Ω_c, and the *design flexural strength*, $\phi_b M_n$, and *allowable flexural strength*, M_n/Ω_b, are determined as follows:

$$\phi_c = 0.75\,(\text{LRFD}) \qquad \Omega_c = 2.00\,(\text{ASD})$$

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

(1) The *nominal strength* of the cross section of a *composite* member subjected to combined axial compression and flexure shall be determined using either the *plastic stress distribution method* or the *strain-compatibility method.*

(2) To account for the influence of length effects on the axial strength of the member, the nominal axial strength of the member shall be determined by Section I2 with P_o taken as the nominal axial strength of the cross section determined in Section I4 (1) above.

I5. SPECIAL CASES

When *composite* construction does not conform to the requirements of Section I1 through Section I4, the strength of *shear connectors* and details of construction shall be established by testing.

CHAPTER J

DESIGN OF CONNECTIONS

This chapter addresses connecting elements, connectors, and the affected elements of the connected members not subject to fatigue *loads*.

The chapter is organized as follows:

User Note: For cases not included in this chapter, the following sections apply:
- Chapter K. Design of HSS and Box Member Connections
- Appendix 3. Design for Fatigue

J1. GENERAL PROVISIONS

1. Design Basis

The *design strength*, ϕR_n, and the *allowable strength* R_n/Ω, of *connections* shall be determined in accordance with the provisions of this chapter and the provisions of Chapter B.

The *required strength* of the connections shall be determined by *structural analysis* for the specified *design loads*, consistent with the type of construction specified, or shall be a proportion of the required strength of the connected members when so specified herein.

Where the gravity axes of intersecting axially loaded members do not intersect at one point, the effects of eccentricity shall be considered.

2. Simple Connections

Simple connections of *beams*, girders, or trusses shall be designed as flexible and are permitted to be proportioned for the reaction shears only, except as otherwise indicated in the design documents. Flexible beam connections shall accommodate end rotations of simple beams. Some inelastic, but self-limiting deformation in the *connection* is permitted to accommodate the end rotation of a simple beam.

3. Moment Connections

End *connections* of restrained *beams*, girders, and trusses shall be designed for the combined effect of *forces* resulting from moment and shear induced by the rigidity of the connections. Response criteria for moment connections are provided in Section B3.6b.

> **User Note:** See Chapter C and Appendix 7 for analysis requirements to establish the *required strength* and *stiffness* for design of *connections*.

4. Compression Members with Bearing Joints

(a) When *column*s bear on bearing plates or are finished to bear at *splices*, there shall be sufficient connectors to hold all parts securely in place.

(b) When compression members other than columns are finished to bear, the splice material and its connectors shall be arranged to hold all parts in line and shall be proportioned for either (i) or (ii) below. It is permissible to use the less severe of the two conditions:

(i) An axial tensile *force* of 50 percent of the required compressive strength of the member; or

(ii) The moment and shear resulting from a transverse *load* equal to 2 percent of the required compressive strength of the member. The transverse load shall be applied at the location of the splice exclusive of other loads that act on the member. The member shall be taken as pinned for the determination of the shears and moments at the splice.

> **User Note:** All compression *joint*s should also be proportioned to resist any tension developed by the *load combinations* stipulated in Section B2.

5. Splices in Heavy Sections

When tensile *forces* due to applied tension or flexure are to be transmitted through *splice*s in heavy sections, as defined in Section A3.1c and A3.1d, by complete-joint-penetration groove (CJP) welds, material notch-toughness requirements as given in Section A3.1c and A3.1d, weld access hole details as given in Section J1.6 and thermal cut surface preparation and inspection requirements as given in M2.2 shall apply. The foregoing provision is not applicable to splices of elements of built-up shapes that are welded prior to assembling the shape.

> **User Note:** CJP groove welded splices of heavy sections can exhibit detrimental effects of weld shrinkage. Members that are sized for compression that are also subject to tensile forces may be less susceptible to damage from shrinkage if they are spliced using PJP groove welds on the flanges and fillet-welded web plates or using bolts for some or all of the splice.

6. Beam Copes and Weld Access Holes

All weld access holes required to facilitate welding operations shall have a length from the toe of the weld preparation not less than 1$^1/_2$ times the thickness of the material in which the hole is made. The height of the access hole shall be 1$^1/_2$ times the thickness of the material with the access hole, t_w, but not less than 1 in. (25 mm) nor does it need to exceed 2 in. (50 mm). The access hole shall be detailed to provide room for weld backing as needed.

For sections that are rolled or welded prior to cutting, the edge of the web shall be sloped or curved from the surface of the flange to the *reentrant* surface of the access hole. In hot-rolled shapes, and built-up shapes with CJP *groove welds* that join the web-to-flange, all *beam copes* and weld access holes shall be free of notches and sharp reentrant corners. No arc of the weld access hole shall have a radius less than $^3/_8$ in. (10 mm).

In built-up shapes with fillet or *partial-joint-penetration groove welds* that join the web-to-flange, all beam copes and weld access holes shall be free of notches and sharp reentrant corners. The access hole shall be permitted to terminate perpendicular to the flange, providing the weld is terminated at least a distance equal to the weld size away from the access hole.

For heavy sections as defined in A3.1c and A3.1d, the *thermally cut* surfaces of beam copes and weld access holes shall be ground to bright metal and inspected by either magnetic particle or dye penetrant methods prior to deposition of *splice* welds. If the curved transition portion of weld access holes and beam copes are formed by predrilled or sawed holes, that portion of the access hole or cope need not be ground. Weld access holes and beam copes in other shapes need not be ground nor inspected by dye penetrant or magnetic particle methods.

7. Placement of Welds and Bolts

Groups of welds or bolts at the ends of any member which transmit axial *force* into that member shall be sized so that the center of gravity of the group coincides with the center of gravity of the member, unless provision is made for the eccentricity. The foregoing provision is not applicable to end connections of statically loaded single angle, double angle, and similar members.

8. Bolts in Combination with Welds

Bolts shall not be considered as sharing the load in combination with welds, except that shear connections with any grade of bolts permitted by Section A3.3 installed in standard holes or short slots transverse to the direction of the load are permitted to be considered to share the load with longitudinally loaded fillet welds. In such connections the available strength of the bolts shall not be taken as greater than 50 percent of the available strength of bearing-type bolts in the connection.

In making welded alterations to structures, existing rivets and high strength bolts tightened to the requirements for *slip-critical connections* are permitted to be utilized for carrying loads present at the time of alteration and the welding need only provide the additional required strength.

9. High-Strength Bolts in Combination with Rivets

In both new work and alterations, in *connections* designed as *slip-critical connections* in accordance with the provisions of Section J3, high-strength bolts are permitted to be considered as sharing the *load* with existing rivets.

10. Limitations on Bolted and Welded Connections

*Pretensioned joint*s, slip-critical *joints* or welds shall be used for the following *connections*:

(1) *Column splice*s in all multi-story structures over 125 ft (38 m) in height
(2) Connections of all *beams* and *girders* to columns and any other beams and girders on which the bracing of columns is dependent in structures over 125 ft (38 m) in height
(3) In all structures carrying cranes of over 5-ton (50 kN) capacity: roof truss splices and connections of trusses to columns, column splices, column bracing, knee braces, and crane supports
(4) Connections for the support of machinery and other live *loads* that produce impact or reversal of load

Snug-tightened joints or joints with ASTM A307 bolts shall be permitted except where otherwise specified.

J2. WELDS

All provisions of AWS D1.1 apply under this Specification, with the exception that the provisions of the listed AISC Specification Sections apply under this Specification in lieu of the cited AWS provisions as follows:

AISC Specification Section J1.6 in lieu of AWS D1.1 Section 5.17.1
AISC Specification Section J2.2a in lieu of AWS D1.1 Section 2.3.2
AISC Specification Table J2.2 in lieu of AWS D1.1 Table 2.1
AISC Specification Table J2.5 in lieu of AWS D1.1 Table 2.3
AISC Specification Appendix 3, Table A-3.1 in lieu of AWS D1.1 Table 2.4
AISC Specification Section B3.9 and Appendix 3 in lieu of AWS D1.1 Section 2, Part C
AISC Specification Section M2.2 in lieu of AWS D1.1 Sections 5.15.4.3 and 5.15.4.4

1. Groove Welds

1a. Effective Area

The effective area of *groove welds* shall be considered as the length of the weld times the effective throat thickness.

The effective throat thickness of a *complete-joint-penetration (CJP) groove weld* shall be the thickness of the thinner part joined.

TABLE J2.1
Effective Throat of Partial-Joint-Penetration Groove Welds

Welding Process	Welding Position F (flat), H (horiz.), V (vert.), OH (overhead)	Groove Type (AWS D1.1, Figure 3.3)	Effective Throat
Shielded Metal Arc (SMAW)	All	J or U Groove 60° V	Depth of Groove
Gas Metal Arc (GMAW) Flux Cored Arc (FCAW)	All		
Submerged Arc (SAW)	F	J or U Groove 60° Bevel or V	
Gas Metal Arc (GMAW) Flux Cored Arc (FCAW)	F, H	45° Bevel	Depth of Groove
Shielded Metal Arc (SMAW)	All	45° Bevel	Depth of Groove Minus $^1/_8$ in. (3 mm)
Gas Metal Arc (GMAW) Flux Cored Arc (FCAW)	V, OH	45° Bevel	Depth of Groove Minus $^1/_8$ in. (3 mm)

The effective throat thickness of a *partial-joint-penetration (PJP) groove weld* shall be as shown in Table J2.1.

User Note: The effective throat size of a partial-joint-penetration groove weld is dependent on the process used and the weld position. The contract documents should either indicate the effective throat required or the weld strength required, and the fabricator should detail the *joint* based on the weld process and position to be used to weld the *joint*.

The effective weld size for flare groove welds, when filled flush to the surface of a round bar, a 90° bend in a *formed section*, or rectangular *HSS* shall be as shown in Table J2.2, unless other effective throats are demonstrated by tests. The effective size of flare groove welds filled less than flush shall be as shown in Table J2.2, less the greatest perpendicular dimension measured from a line flush to the base metal surface to the weld surface.

TABLE J2.2
Effective Weld Sizes of Flare Groove Welds

Welding Process	Flare Bevel Groove[a]	Flare V Groove
GMAW and FCAW-G	$^5/_8 R$	$^3/_4 R$
SMAW and FCAW-S	$^5/_{16} R$	$^5/_8 R$
SAW	$^5/_{16} R$	$^1/_2 R$

[a] For Flare Bevel Groove with $R < {}^3/_8$ in. (10 mm) use only reinforcing fillet weld on filled flush joint.
General Note: R = radius of joint surface (can be assumed to be $2t$ for HSS), in. (mm)

TABLE J2.3
Minimum Effective Throat Thickness of Partial-Joint-Penetration Groove Welds

Material Thickness of Thinner Part Joined, in. (mm)	Minimum Effective Throat Thickness,[a] in. (mm)
To $^1/_4$ (6) inclusive	$^1/_8$ (3)
Over $^1/_4$ (6) to $^1/_2$ (13)	$^3/_{16}$ (5)
Over $^1/_2$ (13) to $^3/_4$ (19)	$^1/_4$ (6)
Over $^3/_4$ (19) to $1^1/_2$ (38)	$^5/_{16}$ (8)
Over $1^1/_2$ (38) to $2^1/_4$ (57)	$^3/_8$ (10)
Over $2^1/_4$ (57) to 6 (150)	$^1/_2$ (13)
Over 6 (150)	$^5/_8$ (16)

[a]See Table J2.1.

Larger effective throat thicknesses than those in Table J2.2 are permitted, provided the fabricator can establish by qualification the consistent production of such larger effective throat thicknesses. Qualification shall consist of sectioning the weld normal to its axis, at mid-length and terminal ends. Such sectioning shall be made on a number of combinations of material sizes representative of the range to be used in the fabrication.

1b. Limitations

The minimum effective throat thickness of a partial-joint-penetration groove weld shall not be less than the size required to transmit calculated *forces* nor the size shown in Table J2.3. Minimum weld size is determined by the thinner of the two parts joined.

2. Fillet Welds

2a. Effective Area

The effective area of a *fillet weld* shall be the *effective length* multiplied by the effective throat. The effective throat of a fillet weld shall be the shortest distance from the root to the face of the diagrammatic weld. An increase in effective throat is permitted if consistent penetration beyond the root of the diagrammatic weld is demonstrated by tests using the production process and procedure variables.

For fillet welds in holes and slots, the effective length shall be the length of the centerline of the weld along the center of the plane through the throat. In the case of overlapping fillets, the effective area shall not exceed the nominal cross-sectional area of the hole or slot, in the plane of the *faying surface*.

2b. Limitations

The minimum size of fillet welds shall be not less than the size required to transmit calculated *forces*, nor the size as shown in Table J2.4. These provisions do not apply to *fillet weld reinforcements* of *partial-* or *complete-joint-penetration groove welds*.

TABLE J2.4
Minimum Size of Fillet Welds

Material Thickness of Thinner Part Joined, in. (mm)	Minimum Size of Fillet Weld,[a] in. (mm)
To 1/4 (6) inclusive	1/8 (3)
Over 1/4 (6) to 1/2 (13)	3/16 (5)
Over 1/2 (13) to 3/4 (19)	1/4 (6)
Over 3/4 (19)	5/16 (8)

[a] Leg dimension of fillet welds. Single pass welds must be used.
Note: See Section J2.2b for maximum size of fillet welds.

The maximum size of fillet welds of connected parts shall be:

(a) Along edges of material less than 1/4-in. (6 mm) thick, not greater than the thickness of the material.

(b) Along edges of material 1/4 in. (6 mm) or more in thickness, not greater than the thickness of the material minus 1/16 in. (2 mm), unless the weld is especially designated on the drawings to be built out to obtain full-throat thickness. In the as-welded condition, the distance between the edge of the base metal and the toe of the weld is permitted to be less than 1/16 in. (2 mm) provided the weld size is clearly verifiable.

The minimum effective length of fillet welds designed on the basis of strength shall be not less than four times the nominal size, or else the size of the weld shall be considered not to exceed 1/4 of its effective length. If longitudinal fillet welds are used alone in end connections of flat-bar tension members, the length of each fillet weld shall be not less than the perpendicular distance between them. For the effect of longitudinal fillet weld length in end connections upon the effective area of the connected member, see Section D3.3.

For end-loaded fillet welds with a length up to 100 times the leg dimension, it is permitted to take the effective length equal to the actual length. When the length of the end-loaded fillet weld exceeds 100 times the weld size, the effective length shall be determined by multiplying the actual length by the reduction factor, β,

$$\beta = 1.2 - 0.002(L/w) \leq 1.0 \qquad \text{(J2-1)}$$

where

L = actual length of end-loaded weld, in. (mm)
w = weld leg size, in. (mm)

When the length of the weld exceeds 300 times the leg size, the value of β shall be taken as 0.60.

Intermittent fillet welds are permitted to be used to transfer calculated *stress* across a *joint* or *faying surfaces* when the *required strength* is less than that developed by a continuous fillet weld of the smallest permitted size, and to join components of *built-up members*. The effective length of any segment of intermittent fillet welding shall be not less than four times the weld size, with a minimum of $1^1/_2$ in. (38 mm).

In *lap joints,* the minimum amount of lap shall be five times the thickness of the thinner part joined, but not less than 1 in. (25 mm). Lap joints joining plates or bars subjected to axial *stress* that utilize transverse fillet welds only shall be fillet welded along the end of both lapped parts, except where the deflection of the lapped parts is sufficiently restrained to prevent opening of the joint under maximum loading.

Fillet weld terminations are permitted to be stopped short or extend to the ends or sides of parts or be boxed except as limited by the following:

(1) For lap joints in which one connected part extends beyond an edge of another connected part that is subject to calculated tensile stress, fillet welds shall terminate not less than the size of the weld from that edge.
(2) For *connections* where flexibility of the outstanding elements is required, when *end returns* are used, the length of the return shall not exceed four times the nominal size of the weld nor half the width of the part.
(3) Fillet welds joining *transverse stiffeners* to *plate girder* webs $^3/_4$ in. (19 mm) thick or less shall end not less than four times nor more than six times the thickness of the web from the web toe of the web-to-flange welds, except where the ends of *stiffeners* are welded to the flange.
(4) Fillet welds that occur on opposite sides of a common plane, shall be interrupted at the corner common to both welds.

User Note: Fillet weld terminations should be located approximately one weld size from of the edge of the connection to minimize notches in the base metal. Fillet welds terminated at the end of the joint, other than those connecting stiffeners to girder webs, are not a cause for correction.

Fillet welds in holes or slots are permitted to be used to transmit shear in lap joints or to prevent the buckling or separation of lapped parts and to join components of built-up members. Such fillet welds may overlap, subject to the provisions of Section J2. Fillet welds in holes or slots are not to be considered plug or *slot welds*.

3. Plug and Slot Welds

3a. Effective Area

The effective shearing area of *plug* and *slot welds* shall be considered as the nominal cross-sectional area of the hole or slot in the plane of the *faying surface*.

3b. Limitations

Plug or slot welds are permitted to be used to transmit shear in *lap joints* or to prevent buckling of lapped parts and to join component parts of *built-up members*.

The diameter of the holes for a plug weld shall not be less than the thickness of the part containing it plus $^5/_{16}$ in. (8 mm), rounded to the next larger odd $^1/_{16}$ in. (even mm), nor greater than the minimum diameter plus $^1/_8$ in. (3 mm) or $2^1/_4$ times the thickness of the weld.

The minimum center-to-center spacing of plug welds shall be four times the diameter of the hole.

The length of slot for a slot weld shall not exceed 10 times the thickness of the weld. The width of the slot shall be not less than the thickness of the part containing it plus $^5/_{16}$ in. (8 mm) rounded to the next larger odd $^1/_{16}$ in. (even mm), nor shall it be larger than $2^1/_4$ times the thickness of the weld. The ends of the slot shall be semicircular or shall have the corners rounded to a radius of not less than the thickness of the part containing it, except those ends which extend to the edge of the part.

The minimum spacing of lines of slot welds in a direction transverse to their length shall be four times the width of the slot. The minimum center-to-center spacing in a longitudinal direction on any line shall be two times the length of the slot.

The thickness of plug or slot welds in material $^5/_8$ in. (16 mm) or less in thickness shall be equal to the thickness of the material. In material over $^5/_8$ in. (16 mm) thick, the thickness of the weld shall be at least one-half the thickness of the material but not less than $^5/_8$ in. (16 mm).

4. Strength

The *design strength*, ϕR_n and the *allowable strength*, R_n/Ω, of welds shall be the lower value of the base material and the *weld metal* strength determined according to the *limit states* of *tensile rupture*, *shear rupture* or *yielding* as follows:

For the base metal

$$R_n = F_{BM} A_{BM} \tag{J2-2}$$

For the weld metal

$$R_n = F_w A_w \tag{J2-3}$$

where

F_{BM} = *nominal strength* of the base metal per unit area, ksi (MPa)
F_w = nominal strength of the weld metal per unit area, ksi (MPa)
A_{BM} = cross-sectional area of the base metal, in.^{2} (mm^2)
A_w = effective area of the weld, in.^{2} (mm^2)

The values of ϕ, Ω, F_{BM}, and F_w and limitations thereon are given in Table J2.5.

TABLE J2.5
Available Strength of Welded Joints, kips (N)

Load Type and Direction Relative to Weld Axis	Pertinent Metal	ϕ and Ω	Nominal Strength (F_{BM} or F_w) kips (N)	Effective Area (A_{BM} or A_w) in.^{2} (mm^2)	Required Filler Metal Strength Level[a][b]
COMPLETE-JOINT-PENETRATION GROOVE WELDS					
Tension Normal to weld axis	Strength of the joint is controlled by the base metal				Matching filler metal shall be used. For T and corner joints with backing left in place, notch tough filler metal is required. See Section J2.6.
Compression Normal to weld axis	Strength of the joint is controlled by the base metal				Filler metal with a strength level equal to or one strength level less than matching filler metal is permitted.
Tension or Compression Parallel to weld axis	Tension or compression in parts joined parallel to a weld need not be considered in design of welds joining the parts.				Filler metal with a strength level equal to or less than matching filler metal is permitted.
Shear	Strength of the joint is controlled by the base metal				Matching filler metal shall be used.[c]
PARTIAL-JOINT-PENETRATION GROOVE WELDS INCLUDING FLARE VEE GROOVE AND FLARE BEVEL GROOVE WELDS					
Tension Normal to weld axis	Base	$\phi = 0.90$ $\Omega = 1.67$	F_y	See J4	Filler metal with a strength level equal to or less than matching filler metal is permitted.
	Weld	$\phi = 0.80$ $\Omega = 1.88$	$0.60F_{EXX}$	See J2.1a	
Compression Column to Base Plate and column splices designed per J1.4(a)	Compressive stress need not be considered in design of welds joining the parts.				
Compression Connections of members designed to bear other than columns as described in J1.4(b)	Base	$\phi = 0.90$ $\Omega = 1.67$	F_y	See J4	
	Weld	$\phi = 0.80$ $\Omega = 1.88$	$0.60F_{EXX}$	See J2.1a	
Compression Connections not finished-to-bear	Base	$\phi = 0.90$ $\Omega = 1.67$	F_y	See J4	
	Weld	$\phi = 0.80$ $\Omega = 1.88$	$0.90F_{EXX}$	See J2.1a	
Tension or Compression Parallel to weld axis	Tension or compression in parts joined parallel to a weld need not be considered in design of welds joining the parts.				
Shear	Base	Governed by J4			
	Weld	$\phi = 0.75$ $\Omega = 2.00$	$0.60F_{EXX}$	See J2.1a	

TABLE J2.5 (cont.)
Available Strength of Welded Joints, kips (N)

Load Type and Direction Relative to Weld Axis	Pertinent Metal	ϕ and Ω	Nominal Strength (F_{bm} or F_w) kips (N)	Effective Area (A_{BM} or A_w) in.^{2} (mm^2)	Required Filler Metal Strength Level[a][b]
FILLET WELDS INCLUDING FILLETS IN HOLES AND SLOTS AND SKEWED T-JOINTS					
Shear	Base	Governed by J4			Filler metal with a strength level equal to or less than matching filler metal is permitted.
	Weld	$\phi = 0.75$ $\Omega = 2.00$	$0.60F_{EXX}$ [d]	See J2.2a	
Tension or Compression Parallel to weld axis	Tension or compression in parts joined parallel to a weld need not be considered in design of welds joining the parts.				
PLUG AND SLOT WELDS					
Shear Parallel to faying surface on the effective area	Base	Governed by J4			Filler metal with a strength level equal to or less than matching filler metal is permitted.
	Weld	$\phi = 0.75$ $\Omega = 2.00$	$0.60F_{EXX}$	J2.3a	

[a] For matching weld metal see AWS D1.1, Section 3.3.
[b] Filler metal with a strength level one strength level greater than matching is permitted.
[c] Filler metals with a strength level less than matching may be used for groove welds between the webs and flanges of built-up sections transferring shear loads, or in applications where high restraint is a concern. In these applications, the weld joint shall be detailed and the weld shall be designed using the thickness of the material as the effective throat, $\phi = 0.80$, $\Omega = 1.88$ and $0.60\ F_{EXX}$ as the nominal strength.
[d] Alternatively, the provisions of J2.4(a) are permitted provided the deformation compatibility of the various weld elements is considered. Alternatively, Sections J2.4(b) and (c) are special applications of J2.4(a) that provide for deformation compatibility.

Alternatively, for *fillet welds* loaded in-plane the *design strength*, ϕR_n and the *allowable strength*, R_n/Ω, of welds is permitted to be determined as follows:

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

(a) For a linear weld group loaded in-plane through the center of gravity

$$R_n = F_w A_w \tag{J2-4}$$

where

$$F_w = 0.60F_{EXX}\left(1.0 + 0.50\sin^{1.5}\theta\right) \tag{J2-5}$$

and

F_{EXX} = electrode classification number, ksi (MPa)
θ = angle of loading measured from the weld longitudinal axis, degrees
A_w = effective area of the weld, in.^{2} (mm^2)

User Note: A linear weld group is one in which all elements are in a line or are parallel.

(b) For weld elements within a weld group that are loaded in-plane and analyzed using an instantaneous center of rotation method, the components of the *nominal strength*, R_{nx} and R_{ny}, are permitted to be determined as follows:

$$R_{nx} = \sum F_{wix} A_{wi} \qquad R_{ny} = \sum F_{wiy} A_{wi} \tag{J2-6}$$

where

A_{wi} = effective area of weld throat of any ith weld element, in.^{2} (mm^2)

$$F_{wi} = 0.60 F_{EXX} \left(1.0 + 0.50 \sin^{1.5}\theta\right) f(p) \tag{J2-7}$$

$$f(p) = [p(1.9 - 0.9p)]^{0.3} \tag{J2-8}$$

F_{wi} = nominal *stress* in any ith weld element, ksi (MPa)

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

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

p = Δ_i/Δ_m, ratio of element i deformation to its deformation at maximum stress

w = weld leg size, in. (mm)

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

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

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

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

(c) For *fillet weld* groups concentrically loaded and consisting of elements that are oriented both longitudinally and transversely to the direction of applied *load*, the combined strength, R_n, of the fillet weld group shall be determined as the greater of

$$R_n = R_{wl} + R_{wt} \tag{J2-9a}$$

or

$$R_n = 0.85 R_{wl} + 1.5 R_{wt} \tag{J2-9b}$$

where

R_{wl} = the total nominal strength of longitudinally loaded fillet welds, as determined in accordance with Table J2.5, kips (N)

R_{wt} = the total nominal strength of transversely loaded fillet welds, as determined in accordance with Table J2.5 without the alternate in Section J2.4(a), kips (N)

5. Combination of Welds

If two or more of the general types of welds (groove, fillet, plug, slot) are combined in a single *joint*, the strength of each shall be separately computed with reference to the axis of the group in order to determine the strength of the combination.

6. Filler Metal Requirements

The choice of electrode for use with *complete-joint-penetration groove weld*s subject to tension normal to the effective area shall comply with the requirements for matching *filler metals* given in AWS D1.1.

User Note: The following User Note Table summarizes the AWS D1.1 provisions for matching filler metals. Other restrictions exist. For a complete list of base metals and prequalified matching filler metals see AWS D1.1, Table 3.1.

Base Metal		Matching Filler Metal
A36 $\leq$ 3/4 in. thick		60 & 70 ksi Electrodes
A36 $>$ 3/4 in. A588* A1011	A572 (Gr. 50 & 55) A913 (Gr. 50) A992 A1018	SMAW: E7015, E7016, E7018, E7028 Other processes: 70 ksi electrodes
A913 (Gr. 60 & 65)		80 ksi electrodes

* For corrosion resistance and color similar to the base see AWS D1.1, Sect. 3.7.3

Notes:

1. Electrodes shall meet the requirements of AWS A5.1, A5.5, A5.17, A5.18, A5.20, A5.23, A5.28 and A5.29.
2. In joints with base metals of different strengths use either a filler metal that matches the higher strength base metal or a filler metal that matches the lower strength and produces a low hydrogen deposit.

Filler metal with a specified *Charpy V-Notch* (CVN) toughness of 20 ft-lbs (27 J) at 40 °F (4 °C) shall be used in the following *joints*:

(1) Complete-joint-penetration groove welded T and corner joints with steel backing left in place, subject to tension normal to the effective area, unless the joints are designed using the *nominal strength* and *resistance factor* or *safety factor* as applicable for a PJP weld.

(2) Complete-joint-penetration groove welded splices subject to tension normal to the effective area in heavy sections as defined in A3.1c and A3.1d.

The manufacturer's Certificate of Conformance shall be sufficient evidence of compliance.

7. Mixed Weld Metal

When Charpy V-Notch toughness is specified, the process consumables for all *weld metal*, tack welds, root pass and subsequent passes deposited in a *joint* shall be compatible to ensure notch-tough composite weld metal.

J3. BOLTS AND THREADED PARTS

1. High-Strength Bolts

Use of high-strength bolts shall conform to the provisions of the *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, hereafter referred to as the RCSC Specification, as approved by the Research Council on Structural Connections, except as otherwise provided in this Specification.

When assembled, all *joint* surfaces, including those adjacent to the washers, shall be free of scale, except tight *mill scale*. All ASTM A325 or A325M and A490

TABLE J3.1
Minimum Bolt Pretension, kips*

Bolt Size, in.	A325 Bolts	A490 Bolts
1/2	12	15
5/8	19	24
3/4	28	35
7/8	39	49
1	51	64
1 1/8	56	80
1 1/4	71	102
1 3/8	85	121
1 1/2	103	148

*Equal to 0.70 times the minimum *tensile strength* of bolts, rounded off to nearest kip, as specified in ASTM specifications for A325 and A490 bolts with UNC threads.

TABLE J3.1M
Minimum Bolt Pretension, kN*

Bolt Size, mm	A325M Bolts	A490M Bolts
M16	91	114
M20	142	179
M22	176	221
M24	205	257
M27	267	334
M30	326	408
M36	475	595

*Equal to 0.70 times the minimum *tensile strength* of bolts, rounded off to nearest kN, as specified in ASTM specifications for A325M and A490M bolts with UNC threads.

or A490M bolts shall be tightened to a bolt tension not less than that given in Table J3.1 or J3.1M, except as noted below. Except as permitted below, installation shall be assured by any of the following methods: turn-of-nut method, a direct tension indicator, calibrated wrench or alternative design bolt.

Bolts are permitted to be installed to only the snug-tight condition when used in

(a) *bearing-type connections.*

(b) tension or combined shear and tension applications, for ASTM A325 or A325M bolts only, where loosening or *fatigue* due to vibration or *load* fluctuations are not design considerations.

The snug-tight condition is defined as the tightness attained by either a few impacts of an impact wrench or the full effort of a worker with an ordinary spud wrench that brings the connected plies into firm contact. Bolts to be tightened only to the snug-tight condition shall be clearly identified on the design and erection drawings.

When ASTM A490 or A490M bolts over 1 in. (25 mm) in diameter are used in slotted or oversized holes in external plies, a single hardened washer conforming to ASTM F436, except with 5/16-in. (8 mm) minimum thickness, shall be used in lieu of the standard washer.

TABLE J3.2
Nominal Stress of Fasteners and Threaded Parts, ksi (MPa)

Description of Fasteners	Nominal Tensile Stress, F_{nt}, ksi (MPa)	Nominal Shear Stress in Bearing-Type Connections, F_{nv}, ksi (MPa)
A307 bolts	45 (310) [a][b]	24 (165) [b][c][f]
A325 or A325M bolts, when threads are not excluded from shear planes	90 (620) [e]	48 (330) [f]
A325 or A325M bolts, when threads are excluded from shear planes	90 (620) [e]	60 (414) [f]
A490 or A490M bolts, when threads are not excluded from shear planes	113 (780) [e]	60 (414) [f]
A490 or A490M bolts, when threads are excluded from shear planes	113 (780) [e]	75 (520) [f]
Threaded parts meeting the requirements of Section A3.4, when threads are not excluded from shear planes	0.75 F_u [a][d]	0.40 F_u
Threaded parts meeting the requirements of Section A3.4, when threads are excluded from shear planes	0.75 F_u [a][d]	0.50 F_u

[a] Subject to the requirements of Appendix 3.

[b] For A307 bolts the tabulated values shall be reduced by 1 percent for each $^1/_{16}$ in. (2 mm) over 5 diameters of length in the grip.

[c] Threads permitted in shear planes.

[d] The nominal tensile strength of the threaded portion of an upset rod, based upon the cross-sectional area at its major thread diameter, A_D, which shall be larger than the nominal body area of the rod before upsetting times F_y.

[e] For A325 or A325M and A490 or A490M bolts subject to tensile fatigue loading, see Appendix 3.

[f] When bearing-type connections used to splice tension members have a fastener pattern whose length, measured parallel to the line of force, exceeds 50 in. (1270 mm), tabulated values shall be reduced by 20 percent.

User Note: Washer requirements are provided in the RCSC Specification, Section 6.

In *slip-critical connections* in which the direction of loading is toward an edge of a connected part, adequate available bearing strength shall be provided based upon the applicable requirements of Section J3.10.

When bolt requirements cannot be provided by ASTM A325 and A325M, F1852, or A490 and A490M bolts because of requirements for lengths exceeding 12 diameters or diameters exceeding $1^1/_2$ in. (38 mm), bolts or threaded rods conforming to ASTM A354 Gr. BC, A354 Gr. BD, or A449 are permitted to be used in accordance with the provisions for threaded rods in Table J3.2.

TABLE J3.3
Nominal Hole Dimensions, in.

Bolt Diameter	Hole Dimensions: Standard (Dia.)	Oversize (Dia.)	Short-Slot (Width × Length)	Long-slot (Width × Length)
$1/2$	$9/16$	$5/8$	$9/16 \times 11/16$	$9/16 \times 1\,1/4$
$5/8$	$11/16$	$13/16$	$11/16 \times 7/8$	$11/16 \times 1\,9/16$
$3/4$	$13/16$	$15/16$	$13/16 \times 1$	$13/16 \times 1\,7/8$
$7/8$	$15/16$	$1\,1/16$	$15/16 \times 1\,1/8$	$15/16 \times 2\,3/16$
1	$1\,1/16$	$1\,1/4$	$1\,1/16 \times 1\,5/16$	$1\,1/16 \times 2\,1/2$
$\geq 1\,1/8$	$d + 1/16$	$d + 5/16$	$(d + 1/16) \times (d + 3/8)$	$(d + 1/16) \times (2.5 \times d)$

TABLE J3.3M
Nominal Hole Dimensions, mm

Bolt Diameter	Hole Dimensions: Standard (Dia.)	Oversize (Dia.)	Short-Slot (Width × Length)	Long-Slot (Width × Length)
M16	18	20	18×22	18×40
M20	22	24	22×26	22×50
M22	24	28	24×30	24×55
M24	27 [a]	30	27×32	27×60
M27	30	35	30×37	30×67
M30	33	38	33×40	33×75
$\geq$M36	$d + 3$	$d + 8$	$(d + 3) \times (d + 10)$	$(d + 3) \times 2.5d$

[a] Clearance provided allows the use of a 1-in. bolt if desirable.

When ASTM A354 Gr. BC, A354 Gr. BD, or A449 bolts and threaded rods are used in slip-critical connections, the bolt geometry including the head and nut(s) shall be equal to or (if larger in diameter) proportional to that provided by ASTM A325 and A325M, or ASTM A490 and A490M bolts. Installation shall comply with all applicable requirements of the RCSC Specification with modifications as required for the increased diameter and/ or length to provide the design pretension.

2. Size and Use of Holes

The maximum sizes of holes for bolts are given in Table J3.3 or J3.3M, except that larger holes, required for tolerance on location of anchor rods in concrete foundations, are permitted in *column* base details.

Standard holes or *short-slotted holes* transverse to the direction of the *load* shall be provided in accordance with the provisions of this specification, unless oversized holes, short-slotted holes parallel to the load or *long-slotted holes* are approved by the *engineer of record*. Finger *shims* up to $1/4$ in. (6 mm) are permitted in *slip-critical connections* designed on the basis of standard holes without reducing the nominal shear strength of the *fastener* to that specified for slotted holes.

Oversized holes are permitted in any or all plies of slip-critical connections, but they shall not be used in *bearing-type connections*. Hardened washers shall be installed over oversized holes in an outer ply.

Short-slotted holes are permitted in any or all plies of slip-critical or bearing-type connections. The slots are permitted without regard to direction of loading in slip-critical connections, but the length shall be normal to the direction of the load in bearing-type connections. Washers shall be installed over short-slotted holes in an outer ply; when high-strength bolts are used, such washers shall be hardened.

Long-slotted holes are permitted in only one of the connected parts of either a slip-critical or bearing-type connection at an individual *faying surface*. Long-slotted holes are permitted without regard to direction of loading in slip-critical connections, but shall be normal to the direction of load in bearing-type connections. Where long-slotted holes are used in an outer ply, plate washers, or a continuous bar with standard holes, having a size sufficient to completely cover the slot after installation, shall be provided. In high-strength bolted connections, such plate washers or continuous bars shall be not less than 5/16 in. (8 mm) thick and shall be of structural grade material, but need not be hardened. If hardened washers are required for use of high-strength bolts, the hardened washers shall be placed over the outer surface of the plate washer or bar.

3. Minimum Spacing

The distance between centers of standard, oversized, or slotted holes, shall not be less than 2 2/3 times the nominal diameter, d, of the *fastener*; a distance of $3d$ is preferred.

4. Minimum Edge Distance

The distance from the center of a standard hole to an edge of a connected part in any direction shall not be less than either the applicable value from Table J3.4 or J3.4M, or as required in Section J3.10. The distance from the center of an oversized or slotted hole to an edge of a connected part shall be not less than that required for a standard hole to an edge of a connected part plus the applicable increment C_2 from Table J3.5 or J3.5M.

> **User Note:** The edge distances in Tables J3.4 and J3.4M are minimum edge distances based on standard fabrication practices and workmanship tolerances. The appropriate provisions of Sections J3.10 and J4 must be satisfied.

5. Maximum Spacing and Edge Distance

The maximum distance from the center of any bolt or rivet to the nearest edge of parts in contact shall be 12 times the thickness of the connected part under consideration, but shall not exceed 6 in. (150 mm). The longitudinal spacing of fasteners between elements in continuous contact consisting of a plate and a shape or two plates shall be as follows:

TABLE J3.4
Minimum Edge Distance,[a] in., from Center of Standard Hole[b] to Edge of Connected Part

Bolt Diameter (in.)	At Sheared Edges	At Rolled Edges of Plates, Shapes or Bars, or Thermally Cut Edges [c]
1/2	7/8	3/4
5/8	1 1/8	7/8
3/4	1 1/4	1
7/8	1 1/2 [d]	1 1/8
1	1 3/4 [d]	1 1/4
1 1/8	2	1 1/2
1 1/4	2 1/4	1 5/8
Over 1 1/4	1 3/4 × d	1 1/4 × d

[a] Lesser edge distances are permitted to be used provided provisions of Section J3.10, as appropriate, are satisfied.

[b] For oversized or slotted holes, see Table J3.5.

[c] All edge distances in this *column* are permitted to be reduced 1/8 in. when the hole is at a point where *required strength* does not exceed 25 percent of the maximum strength in the element.

[d] These are permitted to be 1 1/4 in. at the ends of *beam connection* angles and shear end plates.

TABLE J3.4M
Minimum Edge Distance,[a] mm, from Center of Standard Hole[b] to Edge of Connected Part

Bolt Diameter (mm)	At Sheared Edges	At Rolled Edges of Plates, Shapes or Bars, or Thermally Cut Edges [c]
16	28	22
20	34	26
22	38 [d]	28
24	42 [d]	30
27	48	34
30	52	38
36	64	46
Over 36	1.75*d*	1.25*d*

[a] Lesser edge distances are permitted to be used provided provisions of Section J3.10, as appropriate, are satisfied.
[b] For oversized or slotted holes, see Table J3.5M.
[c] All edge distances in this column are permitted to be reduced 3 mm when the hole is at a point where required strength does not exceed 25 percent of the maximum strength in the element.
[d] These are permitted to be 32 mm at the ends of beam connection angles and shear end plates.

TABLE J3.5
Values of Edge Distance Increment C_2, in.

Nominal Diameter of Fastener (in.)	Oversized Holes	Slotted Holes		
		Long Axis Perpendicular to Edge		Long Axis Parallel to Edge
		Short Slots	Long Slots[a]	
$\leq 7/8$	$1/16$	$1/8$	$3/4d$	0
1	$1/8$	$1/8$		
$\geq 1\,1/8$	$1/8$	$3/16$		

[a]When length of slot is less than maximum allowable (see Table J3.3), C_2 is permitted to be reduced by one-half the difference between the maximum and actual slot lengths.

TABLE J3.5M
Values of Edge Distance Increment C_2, mm

Nominal Diameter of Fastener (mm)	Oversized Holes	Slotted Holes		
		Long Axis Perpendicular to Edge		Long Axis Parallel to Edge
		Short Slots	Long Slots[a]	
≤ 22	2	3	$0.75d$	0
24	3	3		
≥ 27	3	5		

[a]When length of slot is less than maximum allowable (see Table J3.3M), C_2 is permitted to be reduced by one-half the difference between the maximum and actual slot lengths.

(a) For painted members or unpainted members not subject to corrosion, the spacing shall not exceed 24 times the thickness of the thinner plate or 12 in. (305 mm).

(b) For unpainted members of *weathering steel* subject to atmospheric corrosion, the spacing shall not exceed 14 times the thickness of the thinner plate or 7 in. (180 mm).

6. Tension and Shear Strength of Bolts and Threaded Parts

The *design tension* or *shear strength*, ϕR_n, and the *allowable tension* or *shear strength*, R_n/Ω, of a snug-tightened or pretensioned high-strength bolt or threaded part shall be determined according to the *limit states* of *tensile rupture* and *shear rupture* as follows:

$$R_n = F_n A_b \tag{J3-1}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

where

F_n = nominal tensile stress F_{nt}, or shear stress, F_{nv} from Table J3.2, ksi (MPa)

A_b = nominal unthreaded body area of bolt or threaded part (for upset rods, see footnote d, Table J3.2), in.^{2} (mm^2)

The required *tensile strength* shall include any tension resulting from *prying action* produced by deformation of the connected parts.

7. Combined Tension and Shear in Bearing-Type Connections

The *available tensile strength* of a bolt subjected to combined tension and shear shall be determined according to the *limit states* of *tension* and *shear rupture* as follows:

$$R_n = F'_{nt} A_b \tag{J3-2}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

where

F'_{nt} = nominal tensile stress modified to include the effects of shearing *stress*, ksi (MPa)

$$F'_{nt} = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v \le F_{nt} \text{ (LRFD)} \tag{J3-3a}$$

$$F'_{nt} = 1.3F_{nt} - \frac{\Omega F_{nt}}{F_{nv}} f_v \le F_{nt} \text{ (ASD)} \tag{J3-3b}$$

F_{nt} = nominal tensile stress from Table J3.2, ksi (MPa)
F_{nv} = nominal shear stress from Table J3.2, ksi (MPa)
f_v = the required shear stress, ksi (MPa)

The available shear stress of the *fastener* shall equal or exceed the required shear strength per unit area, f_v.

User Note: Note that when the required *stress*, f, in either shear or tension, is less than or equal to 20 percent of the corresponding available stress, the effects of combined *stress* need not be investigated. Also note that Equations J3-3a and J3-3b can be rewritten so as to find a nominal shear stress, F'_{nv}, as a function of the required tensile *stress*, f_t.

8. High-Strength Bolts in Slip-Critical Connections

High-strength bolts in *slip-critical connections* are permitted to be designed to prevent *slip* either as a serviceability limit state or at the required strength limit state. The connection must also be checked for shear strength in accordance with Sections J3.6 and J3.7 and bearing strength in accordance with Sections J3.1 and J3.10.

Slip-critical connections shall be designed as follows, unless otherwise designated by the *engineer of record*. Connections with standard holes or slots transverse to the direction of the load shall be designed for slip as a serviceability limit state. Connections with oversized holes or slots parallel to the direction of the load shall be designed to prevent slip at the required strength level.

The design slip resistance, ϕR_n, and the allowable slip resistance, R_n/Ω, shall be determined for the *limit state* of slip as follows:

$$R_n = \mu D_u h_{sc} T_b N_s \tag{J3-4}$$

For connections in which prevention of slip is a serviceability limit state

$$\phi = 1.00\,(\text{LRFD}) \qquad \Omega = 1.50\,(\text{ASD})$$

For connections designed to prevent slip at the *required strength* level

$$\phi = 0.85\,(\text{LRFD}) \qquad \Omega = 1.76\,(\text{ASD})$$

where

μ = mean slip coefficient for Class A or B surfaces, as applicable, or as established by tests
= 0.35 for Class A surfaces (unpainted clean mill scale steel surfaces or surfaces with Class A coatings on blast-cleaned steel and hot-dipped galvanized and roughened surfaces)
= 0.50 for Class B surfaces (unpainted blast-cleaned steel surfaces or surfaces with Class B coatings on blast-cleaned steel)

D_u = 1.13; a multiplier that reflects the ratio of the mean installed bolt pretension to the specified minimum bolt pretension. The use of other values may be approved by the *engineer of record*.

h_{sc} = hole factor determined as follows:

(a) For standard size holes	$h_{sc} = 1.00$
(b) For oversized and short-slotted holes	$h_{sc} = 0.85$
(c) For long-slotted holes	$h_{sc} = 0.70$

N_s = number of slip planes

T_b = minimum fastener tension given in Table J3.1, kips, or J3.1M, kN

User Note: There are special cases where, with oversize holes and slots parallel to the load, the movement possible due to connection slip could cause a structural failure. Resistance and safety factors are provided for connections where slip is prevented until the required strength load is reached.

Design loads are used for either design method and all connections must be checked for strength as bearing-type connections.

9. Combined Tension and Shear in Slip-Critical Connections

When a *slip-critical connection* is subjected to an applied tension that reduces the net clamping *force*, the available *slip* resistance per bolt, from Section J3.8, shall be multiplied by the factor, k_s, as follows:

$$k_s = 1 - \frac{T_u}{D_u T_b N_b} \qquad (\text{LRFD}) \qquad \text{(J3-5a)}$$

$$k_s = 1 - \frac{1.5T_a}{D_u T_b N_b} \qquad (\text{ASD}) \qquad \text{(J3-5b)}$$

where

N_b = number of bolts carrying the applied tension

T_a = tension force due to *ASD load combinations*, kips (kN)

T_b = minimum *fastener* tension given in Table J3.1 or J3.1M, kips (kN)

T_u = tension *force* due to *LRFD load combinations*, kips (kN)

10. Bearing Strength at Bolt Holes

The *available bearing strength*, ϕR_n and R_n/Ω, at bolt holes shall be determined for the *limit state* of *bearing* as follows:

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

(a) For a bolt in a *connection* with standard, oversized, and short-slotted holes, independent of the direction of loading, or a long-slotted hole with the slot parallel to the direction of the bearing *force*:

(i) When deformation at the bolt hole at *service load* is a design consideration

$$R_n = 1.2\, L_c t F_u \leq 2.4\, d t F_u \tag{J3-6a}$$

(ii) When deformation at the bolt hole at service load is not a design consideration

$$R_n = 1.5\, L_c t F_u \leq 3.0\, d t F_u \tag{J3-6b}$$

(b) For a bolt in a connection with long-slotted holes with the slot perpendicular to the direction of force:

$$R_n = 1.0\, L_c t F_u \leq 2.0\, d t F_u \tag{J3-6c}$$

(c) For connections made using bolts that pass completely through an unstiffened box member or *HSS*, see Section J7 and Equation J7-1,

where

d = nominal bolt diameter, in. (mm)
F_u = *specified minimum tensile strength* of the connected material, ksi (MPa)
L_c = clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material, in. (mm)
t = thickness of connected material, in. (mm)

For connections, the bearing resistance shall be taken as the sum of the bearing resistances of the individual bolts.

Bearing strength shall be checked for both bearing-type and *slip-critical connections*. The use of oversized holes and short- and long-slotted holes parallel to the line of force is restricted to slip-critical connections per Section J3.2.

11. Special Fasteners

The *nominal strength* of special fasteners other than the bolts presented in Table J3.2 shall be verified by tests.

12. Tension Fasteners

When bolts or other fasteners in tension are attached to an unstiffened box or *HSS* wall, the strength of the wall shall be determined by rational analysis.

J4. AFFECTED ELEMENTS OF MEMBERS AND CONNECTING ELEMENTS

This section applies to elements of members at *connections* and connecting elements, such as plates, gussets, angles, and brackets.

1. Strength of Elements in Tension

The *design strength*, ϕR_n, and the *allowable strength*, R_n/Ω, of affected and connecting elements loaded in tension shall be the lower value obtained according to the *limit states* of *tensile yielding* and *tensile rupture*.

(a) For tensile yielding of connecting elements:

$$R_n = F_y A_g \tag{J4-1}$$

$$\phi = 0.90\,(\text{LRFD}) \qquad \Omega = 1.67\,(\text{ASD})$$

(b) For tensile rupture of connecting elements:

$$R_n = F_u A_e \tag{J4-2}$$

$$\phi = 0.75\,(\text{LRFD}) \qquad \Omega = 2.00\,(\text{ASD})$$

where

A_e = *effective net area* as defined in Section D3.3, in.^{2} (mm^2); for bolted *splice* plates, $A_e = A_n \leq 0.85A_g$

2. Strength of Elements in Shear

The available shear yield strength of affected and connecting elements in shear shall be the lower value obtained according to the *limit states* of *shear yielding* and *shear rupture*:

(a) For shear yielding of the element:

$$R_n = 0.60 F_y A_g \tag{J4-3}$$

$$\phi = 1.00\,(\text{LRFD}) \qquad \Omega = 1.50\,(\text{ASD})$$

(b) For shear rupture of the element:

$$R_n = 0.6 F_u A_{nv} \tag{J4-4}$$

$$\phi = 0.75\,(\text{LRFD}) \qquad \Omega = 2.00\,(\text{ASD})$$

where

A_{nv} = net area subject to shear, in.^{2} (mm^2)

3. Block Shear Strength

The *available strength* for the *limit state* of *block shear rupture* along a shear failure path or path(s) and a perpendicular tension failure path shall be taken as

$$R_n = 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.6 F_y A_{gv} + U_{bs} F_u A_{nt} \tag{J4-5}$$

$$\phi = 0.75\,(\text{LRFD}) \qquad \Omega = 2.00\,(\text{ASD})$$

where

A_{gv} = gross area subject to shear, in.^{2} (mm^2)
A_{nt} = *net area* subject to tension, in.^{2} (mm^2)
A_{nv} = net area subject to shear, in.^{2} (mm^2)

Where the tension *stress* is uniform, $U_{bs} = 1$; where the tension stress is non-uniform, $U_{bs} = 0.5$.

User Note: The cases where U_{bs} must be taken equal to 0.5 are illustrated in the Commentary.

4. Strength of Elements in Compression

The available strength of connecting elements in compression for the *limit states* of *yielding* and *buckling* shall be determined as follows.

For $KL/r \leq 25$

$$P_n = F_y A_g \tag{J4-6}$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

For $KL/r > 25$ the provisions of Chapter E apply.

J5. FILLERS

In welded construction, any *filler* $1/4$ in. (6 mm) or more in thickness shall extend beyond the edges of the *splice* plate and shall be welded to the part on which it is fitted with sufficient weld to transmit the splice plate *load*, applied at the surface of the filler. The welds joining the splice plate to the filler shall be sufficient to transmit the splice plate load and shall be long enough to avoid overloading the filler along the toe of the weld. Any filler less than $1/4$ in. (6 mm) thick shall have its edges made flush with the edges of the splice plate and the weld size shall be the sum of the size necessary to carry the splice plus the thickness of the filler plate.

When a bolt that carries load passes through fillers that are equal to or less than $1/4$ in. (6 mm) thick, the shear strength shall be used without reduction. When a bolt that carries load passes through fillers that are greater than $1/4$ in. (6 mm) thick, one of the following requirements shall apply:

(1) For fillers that are equal to or less than $3/4$ in. (19 mm) thick, the shear strength of the bolts shall be multiplied by the factor $[1 - 0.4(t - 0.25)]$ [S.I.: $[1 - 0.0154(t - 6)]$], where t is the total thickness of the fillers up to $3/4$ in. (19 mm);
(2) The fillers shall be extended beyond the *joint* and the filler extension shall be secured with enough bolts to uniformly distribute the total *force* in the connected element over the combined cross section of the connected element and the fillers;
(3) The size of the joint shall be increased to accommodate a number of bolts that is equivalent to the total number required in (2) above; or
(4) The joint shall be designed to prevent *slip* at required strength levels in accordance with Section J3.8.

J6. SPLICES

Groove-welded *splices* in *plate girders* and *beams* shall develop the *nominal strength* of the smaller spliced section. Other types of splices in cross sections of plate girders and beams shall develop the strength required by the forces at the point of the splice.

J7. BEARING STRENGTH

The *design bearing strength*, ϕR_n, and the *allowable bearing strength*, R_n/Ω, of surfaces in contact shall be determined for the *limit state* of *bearing (local compressive yielding)* as follows:

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

The nominal bearing strength, R_n, is defined as follows for the various types of bearing:

(a) For *milled surfaces*, pins in reamed, drilled, or bored holes, and ends of *fitted bearing stiffeners*:

$$R_n = 1.8 F_y A_{pb} \tag{J7-1}$$

where

F_y = *specified minimum yield stress*, ksi (MPa)
A_{pb} = projected bearing area, in.^{2} (mm^2)

(b) For *expansion rollers* and rockers:

(i) If $d \leq 25$ in. (635 mm)

$$R_n = 1.2(F_y - 13)ld/20 \tag{J7-2}$$

$$(\text{SI: } R_n = 1.2(F_y - 90)ld/20) \tag{J7-2M}$$

(ii) If $d > 25$ in. (635 mm)

$$R_n = 6.0(F_y - 13)l\sqrt{d}/20 \tag{J7-3}$$

$$(\text{SI: } R_n = 30.2(F_y - 90)l\sqrt{d}/20) \tag{J7-3M}$$

where

d = diameter, in. (mm)
l = length of bearing, in. (mm)

J8. COLUMN BASES AND BEARING ON CONCRETE

Proper provision shall be made to transfer the column loads and moments to the footings and foundations.

In the absence of code regulations, the *design bearing strength*, $\phi_c P_p$, and the *allowable bearing strength*, P_p/Ω_c, for the *limit state* of *concrete crushing* are permitted to be taken as follows:

$$\phi_c = 0.60 \text{ (LRFD)} \qquad \Omega_c = 2.50 \text{ (ASD)}$$

The nominal bearing strength, P_p, is determined as follows:

(a) On the full area of a concrete support:

$$P_p = 0.85 f_c' A_1 \tag{J8-1}$$

(b) On less than the full area of a concrete support:

$$P_p = 0.85 f_c' A_1 \sqrt{A_2/A_1} \le 1.7 f_c' A_1 \tag{J8-2}$$

where

A_1 = area of steel concentrically bearing on a concrete support, in.^{2} (mm^2)
A_2 = maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, in.^{2} (mm^2)

J9. ANCHOR RODS AND EMBEDMENTS

Anchor rods shall be designed to provide the required resistance to loads on the completed structure at the base of columns including the net tensile components of any bending moment that may result from load combinations stipulated in Section B2. The anchor rods shall be designed in accordance with the requirements for threaded parts in Table J3.2.

Larger oversized and slotted holes are permitted in base plates when adequate bearing is provided for the nut by using structural or plate washers to bridge the hole.

> **User Note:** The permitted hole sizes and corresponding washer dimensions are given in the AISC *Manual of Steel Construction.*

When horizontal forces are present at *column* bases, these forces should, where possible, be resisted by bearing against concrete elements or by shear friction between the column base plate and the foundation. When anchor rods are designed to resist horizontal force the base plate hole size, the anchor rod setting tolerance, and the horizontal movement of the column shall be considered in the design.

> **User Note**: See ACI 318 for embedment design and for shear friction design. See OSHA for special erection requirements for anchor rods.

J10. FLANGES AND WEBS WITH CONCENTRATED FORCES

This section applies to *single-* and *double-concentrated forces* applied normal to the flange(s) of wide flange sections and similar built-up shapes. A single-concentrated force can be either tensile or compressive. Double-concentrated forces are one tensile and one compressive and form a couple on the same side of the loaded member.

When the *required strength* exceeds the *available strength* as determined for the *limit states* listed in this section, *stiffeners* and/or *doublers* shall be provided and shall be sized for the difference between the required strength and the available strength for the applicable *limit state*. Stiffeners shall also meet the design requirements in Section J10.8. Doublers shall also meet the design requirement in Section J10.9

User Note: See Appendix 6.3 for requirements for the ends of cantilever members.

Stiffeners are required at *unframed ends* of *beams* in accordance with the requirements of Section J10.7.

1. Flange Local Bending

This section applies to tensile *single-concentrated forces* and the tensile component of *double-concentrated forces*.

The *design strength*, ϕR_n, and the *allowable strength*, R_n/Ω, for the *limit state* of flange *local bending* shall be determined as follows:

$$R_n = 6.25 t_f^{\,2} F_{yf} \tag{J10-1}$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

where

F_{yf} = *specified minimum yield stress* of the flange, ksi (MPa)
t_f = thickness of the loaded flange, in. (mm)

If the length of loading across the member flange is less than $0.15b_f$, where b_f is the member flange width, Equation J10-1 need not be checked.

When the concentrated *force* to be resisted is applied at a distance from the member end that is less than $10t_f$, R_n shall be reduced by 50 percent.

When required, a pair of *transverse stiffeners* shall be provided.

2. Web Local Yielding

This section applies to *single-concentrated forces* and both components of *double-concentrated forces*.

The *available strength* for the *limit state* of web *local yielding* shall be determined as follows:

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

The *nominal strength*, R_n, shall be determined as follows:

(a) When the concentrated *force* to be resisted is applied at a distance from the member end that is greater than the depth of the member d,

$$R_n = (5k + N) F_{yw} t_w \tag{J10-2}$$

(b) When the concentrated force to be resisted is applied at a distance from the member end that is less than or equal to the depth of the member d,

$$R_n = (2.5k + N)F_{yw}t_w \tag{J10-3}$$

where

k = distance from outer face of the flange to the web toe of the fillet, in. (mm)
F_{yw} = *specified minimum yield stress* of the web, ksi (MPa)
N = length of bearing (not less than k for end *beam* reactions), in. (mm)
t_w = web thickness, in. (mm)

When required, a pair of *transverse stiffeners* or a *doubler* plate shall be provided.

3. Web Crippling

This section applies to compressive *single-concentrated forces* or the compressive component of *double-concentrated forces*.

The *available strength* for the *limit state* of *web local crippling* shall be determined as follows:

$$\phi = 0.75\,(\text{LRFD}) \qquad \Omega = 2.00\,(\text{ASD})$$

The *nominal strength*, R_n, shall be determined as follows:

(a) When the concentrated compressive *force* to be resisted is applied at a distance from the member end that is greater than or equal to $d/2$:

$$R_n = 0.80t_w^2\left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}} \tag{J10-4}$$

(b) When the concentrated compressive force to be resisted is applied at a distance from the member end that is less than $d/2$:

(i) For $N/d \leq 0.2$

$$R_n = 0.40t_w^2\left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}} \tag{J10-5a}$$

(ii) For $N/d > 0.2$

$$R_n = 0.40t_w^2\left[1 + \left(\frac{4N}{d} - 0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}} \tag{J10-5b}$$

where

d = overall depth of the member, in. (mm)
t_f = flange thickness, in. (mm)

When required, a *transverse stiffener*, or pair of transverse stiffeners, or a *doubler* plate extending at least one-half the depth of the web shall be provided.

4. Web Sidesway Buckling

This Section applies only to compressive *single-concentrated forces* applied to members where relative lateral movement between the loaded compression flange and the tension flange is not restrained at the point of application of the concentrated *force*.

The *available strength* of the web shall be determined as follows:

$$\phi = 0.85 \text{ (LRFD)} \qquad \Omega = 1.76 \text{ (ASD)}$$

The *nominal strength*, R_n, for the *limit state* of *web sidesway buckling* shall be determined as follows:

(a) If the compression flange is restrained against rotation:

(i) For $(h/t_w)/(l/b_f) \leq 2.3$

$$R_n = \frac{C_r t_w^3 t_f}{h^2}\left[1 + 0.4\left(\frac{h/t_w}{l/b_f}\right)^3\right] \tag{J10-6}$$

(ii) For $(h/t_w)/(l/b_f) > 2.3$, the limit state of web sidesway buckling does not apply.

When the *required strength* of the web exceeds the available strength, local *lateral bracing* shall be provided at the tension flange or either a pair of *transverse stiffeners* or a *doubler* plate shall be provided.

(b) If the compression flange is not restrained against rotation:

(i) For $(h/t_w)/(l/b_f) \leq 1.7$

$$R_n = \frac{C_r t_w^3 t_f}{h^2}\left[0.4\left(\frac{h/t_w}{l/b_f}\right)^3\right] \tag{J10-7}$$

(ii) For $(h/t_w)/(l/b_f) > 1.7$, the limit state of web sidesway buckling does not apply.

When the required strength of the web exceeds the available strength, local *lateral bracing* shall be provided at both flanges at the point of application of the concentrated forces.

In Equations J10-6 and J10-7, the following definitions apply:

b_f = flange width, in. (mm)
C_r = 960,000 ksi (6.62×10^6 MPa) when $M_u < M_y$ (LRFD) or $1.5M_a < M_y$ (ASD) at the location of the force
= 480,000 ksi (3.31×10^6 MPa) when $M_u \geq M_y$ (LRFD) or $1.5M_a \geq M_y$ (ASD) at the location of the force
h = clear distance between flanges less the fillet or corner radius for rolled shapes; distance between adjacent lines of *fasteners* or the clear distance between flanges when welds are used for built-up shapes, in. (mm)
l = largest laterally *unbraced length* along either flange at the point of *load*, in. (mm)
t_f = flange thickness, in. (mm)
t_w = web thickness, in. (mm)

User Note: For determination of adequate restraint, refer to Appendix 6.

5. Web Compression Buckling

This Section applies to a pair of compressive *single-concentrated forces* or the compressive components in a pair of *double-concentrated forces*, applied at both flanges of a member at the same location.

The *available strength* for the *limit state* of web *local buckling* shall be determined as follows:

$$R_n = \frac{24t_w^3\sqrt{EF_{yw}}}{h} \tag{J10-8}$$

$$\phi = 0.90\,(\text{LRFD}) \qquad \Omega = 1.67\,(\text{ASD})$$

When the pair of concentrated compressive *forces* to be resisted is applied at a distance from the member end that is less than $d/2$, R_n shall be reduced by 50 percent.

When required, a single *transverse stiffener*, a pair of transverse stiffeners, or a *doubler* plate extending the full depth of the web shall be provided.

6. Web Panel Zone Shear

This section applies to *double-concentrated forces* applied to one or both flanges of a member at the same location.

The *available strength* of the web *panel zone* for the *limit state* of *shear yielding* shall be determined as follows:

$$\phi = 0.90\ (\text{LRFD}) \qquad \Omega = 1.67\ (\text{ASD})$$

The *nominal strength*, R_n, shall be determined as follows:

(a) When the effect of panel-zone deformation on frame *stability* is not considered in the analysis:

(i) For $P_r \leq 0.4P_c$

$$R_n = 0.60F_y d_c t_w \tag{J10-9}$$

(ii) For $P_r > 0.4P_c$

$$R_n = 0.60F_y d_c t_w \left(1.4 - \frac{P_r}{P_c}\right) \tag{J10-10}$$

(b) When frame stability, including plastic panel-zone deformation, is considered in the analysis:

(i) For $P_r \leq 0.75P_c$

$$R_n = 0.60F_y d_c t_w \left(1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_w}\right) \tag{J10-11}$$

(ii) For $P_r > 0.75P_c$

$$R_n = 0.60F_y d_c t_w \left(1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_w}\right)\left(1.9 - \frac{1.2P_r}{P_c}\right) \qquad \text{(J10-12)}$$

In Equations J10-9 through J10-12, the following definitions apply:

A = *column* cross-sectional area, in.^{2} (mm^2)
b_{cf} = width of column flange, in. (mm)
d_b = *beam* depth, in. (mm)
d_c = column depth, in. (mm)
F_y = *specified minimum yield stress* of the column web, ksi (MPa)
P_c = P_y, kips (N) (LRFD)
P_c = $0.6P_y$, kips (N) (ASD)
P_r = *required strength*, kips (N)
P_y = F_yA, axial *yield strength* of the column, kips (N)
t_{cf} = thickness of the column flange, in. (mm)
t_w = column web thickness, in. (mm)

When required, *doubler* plate(s) or a pair of *diagonal stiffeners* shall be provided within the boundaries of the rigid connection whose webs lie in a common plane.

See Section J10.9 for doubler plate design requirements.

7. Unframed Ends of Beams and Girders

At *unframed ends* of *beams* and *girders* not otherwise restrained against rotation about their longitudinal axes, a pair of *transverse stiffeners*, extending the full depth of the web, shall be provided.

8. Additional Stiffener Requirements for Concentrated Forces

Stiffeners required to resist tensile concentrated *forces* shall be designed in accordance with the requirements of Chapter D and welded to the loaded flange and the web. The welds to the flange shall be sized for the difference between the *required strength* and available *limit state* strength. The stiffener to web welds shall be sized to transfer to the web the algebraic difference in tensile force at the ends of the stiffener.

Stiffeners required to resist compressive concentrated forces shall be designed in accordance with the requirements in Sections E6.2 and J4.4 and shall either bear on or be welded to the loaded flange and welded to the web. The welds to the flange shall be sized for the difference between the required strength and the applicable limit state strength. The weld to the web shall be sized to transfer to the web the algebraic difference in compression force at the ends of the stiffener. For fitted bearing stiffeners, see Section J7.

Transverse full depth bearing stiffeners for compressive forces applied to a *beam* or *plate girder* flange(s) shall be designed as axially compressed members (*columns*) in accordance with the requirements of Sections E6.2 and J4.4.

The member properties shall be determined using an *effective length* of $0.75h$ and a cross section composed of two stiffeners and a strip of the web having a width of $25t_w$ at interior stiffeners and $12t_w$ at the ends of members. The weld connecting full depth bearing stiffeners to the web shall be sized to transmit the difference in compressive force at each of the stiffeners to the web.

Transverse and *diagonal stiffeners* shall comply with the following additional criteria:

(1) The width of each *stiffener* plus one-half the thickness of the column web shall not be less than one-third of the width of the flange or moment connection plate delivering the concentrated force.
(2) The thickness of a stiffener shall not be less than one-half the thickness of the flange or moment connection plate delivering the concentrated *load*, and greater than or equal to the width divided by 15.
(3) *Transverse stiffeners* shall extend a minimum of one-half the depth of the member except as required in J10.5 and J10.7.

9. Additional Doubler Plate Requirements for Concentrated Forces

Doubler plates required for compression strength shall be designed in accordance with the requirements of Chapter E.

Doubler plates required for tensile strength shall be designed in accordance with the requirements of Chapter D.

Doubler plates required for shear strength (see Section J10.6) shall be designed in accordance with the provisions of Chapter G.

In addition, doubler plates shall comply with the following criteria:
(1) The thickness and extent of the doubler plate shall provide the additional material necessary to equal or exceed the strength requirements.
(2) The doubler plate shall be welded to develop the proportion of the total force transmitted to the doubler plate.

CHAPTER K

DESIGN OF HSS AND BOX MEMBER CONNECTIONS

This chapter covers member strength design considerations pertaining to connections to HSS members and box sections of uniform wall thickness. See also Chapter J for additional requirements for bolting to HSS.

The chapter is organized as follows:

User Note: See Section J3.10(c) for through-bolts.

K1. CONCENTRATED FORCES ON HSS

1. Definitions of Parameters

B = overall width of rectangular *HSS* member, measured 90 degrees to the plane of the *connection*, in. (mm)

B_p = width of plate, measured 90 degrees to the plane of the connection, in. (mm)

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

F_y = *specified minimum yield stress* of HSS member material, ksi (MPa)

F_{yp} = specified minimum yield stress of plate, ksi (MPa)

F_u = *specified minimum tensile strength* of HSS material, ksi (MPa)

H = overall height of rectangular HSS member, measured in the plane of the connection, in. (mm)

N = bearing length of the *load*, measured parallel to the axis of the HSS member, (or measured across the width of the HSS in the case of loaded cap plates), in. (mm)

t = *design wall thickness* of HSS member, in. (mm)

t_p = thickness of plate, in. (mm)

2. Limits of Applicability

The criteria herein are applicable only when the *connection* configuration is within the following limits of applicability:

(1) Strength: $F_y \leq 52$ ksi (360 MPa) for *HSS*
(2) Ductility: $F_y/F_u \leq 0.8$ for HSS
(3) Other limits apply for specific criteria

3. Concentrated Force Distributed Transversely

3a. Criterion for Round HSS

When a concentrated *force* is distributed transversely to the axis of the *HSS* the *design strength,* ϕR_n, and the *allowable strength,* R_n/Ω, for the *limit state* of *local yielding* shall be determined as follows:

$$R_n = F_y t^2[5.5/(1 - 0.81B_p/D)]Q_f \tag{K1-1}$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

where Q_f is given by Equation K2-1.

Additional limits of applicability are

(1) $0.2 < B_p/D \leq 1.0$
(2) $D/t \leq 50$ for *T-connection*s and $D/t \leq 40$ for *cross-connections*

3b. Criteria for Rectangular HSS

When a concentrated force is distributed transversely to the axis of the HSS the design strength, ϕR_n, and the allowable strength, R_n/Ω, shall be the lowest value according to the limit states of local yielding due to *uneven load distribution*, *shear yielding (punching)* and sidewall strength.

Additional limits of applicability are

(1) $0.25 < B_p/B \leq 1.0$
(2) B/t for the loaded HSS wall ≤ 35

(a) For the limit state of local yielding due to uneven load distribution in the loaded plate,

$$R_n = [10F_y t/(B/t)]B_p \leq F_{yp} t_p B_p \tag{K1-2}$$

$$\phi = 0.95 \text{ (LRFD)} \qquad \Omega = 1.58 \text{ (ASD)}$$

(b) For the limit state of shear yielding (punching),

$$R_n = 0.6F_y t\ [2t_p + 2B_{ep}] \tag{K1-3}$$

$$\phi = 0.95 \text{ (LRFD)} \qquad \Omega = 1.58 \text{ (ASD)}$$

where

$$B_{ep} = 10B_p/(B/t) \leq B_p$$

This limit state need not be checked when $B_p > (B - 2t)$, nor when $B_p < 0.85B$.

(c) For the limit state of sidewall under tension loading, the available strength shall be taken as the strength for sidewall local yielding. For the limit state of sidewall under compression loading, available strength shall be taken as the lowest value obtained according to the limit states of sidewall local yielding, sidewall local crippling and sidewall local buckling.

This limit state need not be checked unless the *chord member* and *branch member* (connecting element) have the same width ($\beta = 1.0$).

(i) For the limit state of sidewall local yielding,

$$R_n = 2F_y t[5k + N] \quad \text{(K1-4)}$$

$$\phi = 1.0 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

where

k = outside corner radius of the HSS, which is permitted to be taken as $1.5t$ if unknown, in. (mm)

(ii) For the limit state of sidewall local crippling, in T-connections,

$$R_n = 1.6t^2[1 + 3N/(H - 3t)]\,(EF_y)^{0.5} Q_f \quad \text{(K1-5)}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.0 \text{ (ASD)}$$

where Q_f is given by Equation K2-10.

(iii) For the limit state of sidewall local buckling in cross-connections,

$$R_n = [48t^3/(H - 3t)]\,(EF_y)^{0.5} Q_f \quad \text{(K1-6)}$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

where Q_f is given by Equation K2-10.

The nonuniformity of load transfer along the line of weld, due to the flexibility of the HSS wall in a transverse plate-to-HSS connection, shall be considered in proportioning such welds. This requirement can be satisfied by limiting the total effective weld length, L_e, of groove and fillet welds to rectangular HSS as follows:

$$L_e = 2[10/(B/t)]\,[(F_y t)/(F_{yp} t_p)]B_p \le 2B_p \quad \text{(K1-7)}$$

where

L_e = total effective weld length for welds on both sides of the transverse plate, in. (mm)

In lieu of Equation K1-7, this requirement may be satisfied by other rational approaches.

User Note: An upper limit on weld size will be given by the weld that develops the available strength of the connected element.

4. Concentrated Force Distributed Longitudinally at the Center of the HSS Diameter or Width, and Acting Perpendicular to the HSS Axis

When a concentrated *force* is distributed longitudinally along the axis of the *HSS* at the center of the HSS diameter or width, and also acts perpendicular to the axis direction of the HSS (or has a component perpendicular to the axis direction of the

HSS), the *design strength*, ϕR_n, and the *allowable strength*, R_n/Ω, perpendicular to the HSS axis shall be determined for the *limit state* of *chord plastification* as follows.

4a. Criterion for Round HSS

An additional limit of applicability is:
$D/t \leq 50$ for *T-connections* and $D/t \leq 40$ for *cross-connections*

$$R_n = 5.5 F_y t^2 (1 + 0.25N/D) Q_f \tag{K1-8}$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

where Q_f is given by Equation K2-1.

4b. Criterion for Rectangular HSS

An additional limit of applicability is:
B/t for the loaded HSS wall ≤ 40

$$R_n = [F_y t^2/(1 - t_p/B)]\,[2N/B + 4(1 - t_p/B)^{0.5} Q_f] \tag{K1-9}$$

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

where

$Q_f = (1 - U^2)^{0.5}$
U is given by Equation K2-12

5. Concentrated Force Distributed Longitudinally at the Center of the HSS Width, and Acting Parallel to the HSS Axis

When a concentrated *force* is distributed longitudinally along the axis of a rectangular *HSS*, and also acts parallel but eccentric to the axis direction of the member, the *connection* shall be verified as follows:

$$F_{yp} t_p \leq F_u t \tag{K1-10}$$

User Note: This provision is primarily intended for shear tab connections. Equation K1-10 precludes shear yielding (punching) of the HSS wall by requiring the plate (shear tab) strength to be less than the HSS wall strength. For bracing connections to HSS columns, where a load is applied by a longitudinal plate at an angle to the HSS axis, the connection design will be governed by the force component perpendicular to the HSS axis (see Section K1.4b).

6. Concentrated Axial Force on the End of a Rectangular HSS with a Cap Plate

When a concentrated *force* acts on the end of a capped *HSS*, and the force is in the direction of the HSS axis, the *design strength*, ϕR_n, and the *allowable strength*, R_n/Ω, shall be determined for the *limit states* of wall *local yielding* (due to tensile or compressive *forces*) and wall *local crippling* (due to compressive forces only), with consideration for shear lag, as follows.

User Note: The procedure below presumes that the concentrated *force* has a dispersion slope of 2.5:1 through the cap plate (of thickness t_p) and disperses into the two HSS walls of dimension B.

If $(5t_p + N) \geq B$, the *available strength* of the HSS is computed by summing the contributions of all four HSS walls.

If $(5t_p + N) < B$, the available strength of the HSS is computed by summing the contributions of the two walls into which the *load* is distributed.

(i) For the limit state of wall local yielding, for one wall,

$$R_n = F_y t[5t_p + N] \leq BF_y t \tag{K1-11}$$

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

(ii) For the limit state of wall local crippling, for one wall,

$$R_n = 0.8t^2[1 + (6N/B)(t/t_p)^{1.5}][EF_y t_p/t]^{0.5} \tag{K1-12}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

K2. HSS-TO-HSS TRUSS CONNECTIONS

HSS-to-HSS truss *connections* are defined as connections that consist of one or more *branch members* that are directly welded to a continuous chord that passes through the connection and shall be classified as follows:

(a) When the *punching load* ($P_r \sin\theta$) in a branch member is equilibrated by *beam* shear in the *chord member*, the connection shall be classified as a *T-connection* when the branch is perpendicular to the chord and a *Y-connection* otherwise.

(b) When the punching load ($P_r \sin\theta$) in a branch member is essentially equilibrated (within 20 percent) by *loads* in other branch member(s) on the same side of the connection, the connection shall be classified as a *K-connection*. The relevant gap is between the primary branch members whose loads equilibrate. An *N-connection* can be considered as a type of K-connection.

User Note: A K-connection with one branch perpendicular to the chord is often called an N-connection.

(c) When the punching load ($P_r \sin\theta$) is transmitted through the chord member and is equilibrated by branch member(s) on the opposite side, the connection shall be classified as a *cross-connection*.

(d) When a connection has more than two primary branch members, or branch members in more than one plane, the connection shall be classified as a general or multiplanar connection.

When branch members transmit part of their load as K-connections and part of their load as T-, Y-, or cross-connections, the *nominal strength* shall be determined by interpolation on the proportion of each in total.

For the purposes of this Specification, the centerlines of branch members and chord members shall lie in a common plane. Rectangular HSS connections are further limited to have all members oriented with walls parallel to the plane. For trusses that are made with HSS that are connected by welding branch members to chord members, eccentricities within the limits of applicability are permitted without consideration of the resulting moments for the design of the connection.

1. Definitions of Parameters

B = overall width of rectangular HSS main member, measured 90 degrees to the plane of the connection, in. (mm)

B_b = overall width of rectangular HSS branch member, measured 90 degrees to the plane of the connection, in. (mm)

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

D_b = outside diameter of round HSS branch member, in. (mm)

e = eccentricity in a truss connection, positive being away from the branches, in. (mm)

F_y = *specified minimum yield stress* of HSS main member material, ksi (MPa)

F_{yb} = specified minimum yield stress of HSS branch member material, ksi (MPa)

F_u = *specified minimum tensile strength* of HSS material, ksi (MPa)

g = gap between toes of branch members in a gapped K-connection, neglecting the welds, in. (mm)

H = overall height of rectangular HSS main member, measured in the plane of the connection, in. (mm)

H_b = overall height of rectangular HSS branch member, measured in the plane of the connection, in. (mm)

t = *design wall thickness* of HSS main member, in. (mm)

t_b = design wall thickness of HSS branch member, in. (mm)

β = the width ratio; the ratio of branch diameter to chord diameter = D_b/D for round HSS; the ratio of overall branch width to chord width = B_b/B for rectangular HSS

β_{eff} = the *effective width* ratio; the sum of the perimeters of the two branch members in a K-connection divided by eight times the chord width

γ = the chord slenderness ratio; the ratio of one-half the diameter to the wall thickness = $D/2t$ for round HSS; the ratio of one-half the width to wall thickness = $B/2t$ for rectangular HSS

η = the *load* length parameter, applicable only to rectangular *HSS*; the ratio of the length of contact of the branch with the chord in the plane of the connection to the chord width = N/B, where $N = H_b/\sin\theta$

θ = acute angle between the branch and chord (degrees)

ζ = the gap ratio; the ratio of the gap between the branches of a gapped K-connection to the width of the chord = g/B for rectangular HSS

2. Criteria for Round HSS

The interaction of *stress* due to *chord member forces* and local branch connection forces shall be incorporated through the chord-stress interaction parameter Q_f.

When the chord is in tension,

$$Q_f = 1$$

When the chord is in compression,

$$Q_f = 1.0 - 0.3U(1 + U) \tag{K2-1}$$

where U is the utilization ratio given by

$$U = |P_r/A_g F_c + M_r/SF_c| \tag{K2-2}$$

and

P_r = required axial strength in chord, kips (N); for K-connections, P_r is to be determined on the side of the *joint* that has the lower compression stress (lower U)
M_r = required flexural strength in chord, kip-in. (N-mm)
A_g = chord gross area, in.^{2} (mm^2)
F_c = *available stress*, ksi (MPa)
S = chord elastic section modulus, in.^{3} (mm^3)

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

$P_r = P_u$ = required axial strength in chord, using *LRFD load combinations*, kips (N)
$M_r = M_u$ = required flexural strength in chord, using LRFD load combinations, kip-in. (N-mm)
$F_c = F_y$, ksi (MPa)

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

$P_r = P_a$ = required axial strength in chord, using *ASD load combinations*, kips (N)
$M_r = M_a$ = required flexural strength in chord, using ASD load combinations, kip-in. (N-mm)
$F_c = 0.6\, F_y$, ksi (MPa)

2a. Limits of Applicability

The criteria herein are applicable only when the *connection* configuration is within the following limits of applicability:

(1) *Joint eccentricity*: $-0.55D \leq e \leq 0.25D$, where D is the chord diameter and e is positive away from the branches
(2) Branch angle: $\theta \geq 30°$
(3) Chord wall slenderness: ratio of diameter to wall thickness less than or equal to 50 for T-, Y- and *K-connections*; less than or equal to 40 for *cross-connections*

(4) Tension branch wall slenderness: ratio of diameter to wall thickness less than or equal to 50
(5) Compression branch wall slenderness: ratio of diameter to wall thickness less than or equal to $0.05E/F_y$
(6) Width ratio: $0.2 < D_b/D \leq 1.0$ in general, and $0.4 \leq D_b/D \leq 1.0$ for gapped K-connections
(7) If a *gap connection*: g greater than or equal to the sum of the branch wall thicknesses
(8) If an *overlap connection*: $25\% \leq O_v \leq 100\%$, where $O_v = (q/p) \times 100\%$. p is the projected length of the overlapping branch on the chord; q is the overlap length measured along the connecting face of the chord beneath the two branches. For overlap connections, the larger (or if equal diameter, the thicker) branch is a "thru member" connected directly to the chord.
(9) Branch thickness ratio for overlap connections: thickness of overlapping branch to be less than or equal to the thickness of the overlapped branch
(10) Strength: $F_y \leq 52$ ksi (360 MPa) for chord and branches
(11) Ductility: $F_y / F_u \leq 0.8$

2b. Branches with Axial Loads in T-, Y- and Cross-Connections

For T- and Y- connections, the *design strength* of the branch ϕP_n, or the *allowable strength* of the branch, P_n/Ω, shall be the lower value obtained according to the *limit states* of *chord plastification* and *shear yielding (punching)*.

(a) For the limit state of chord plastification in T- and Y-connections,

$$P_n \sin\theta = F_y t^2 [3.1 + 15.6\beta^2]\gamma^{0.2} Q_f \tag{K2-3}$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

(b) For the limit state of shear yielding (punching),

$$P_n = 0.6 F_y t \pi D_b [(1 + \sin\theta)/2\sin^2\theta] \tag{K2-4}$$

$$\phi = 0.95 \text{ (LRFD)} \qquad \Omega = 1.58 \text{ (ASD)}$$

This limit state need not be checked when $\beta > (1 - 1/\gamma)$.

(c) For the limit state of chord plastification in cross-connections,

$$P_n \sin\theta = F_y t^2 [5.7/(1 - 0.81\beta)] Q_f \tag{K2-5}$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

2c. Branches with Axial Loads in K-Connections

For K-connections, the design strength of the branch, ϕP_n, and the allowable strength of the branch, P_n/Ω, shall be the lower value obtained according to the limit states of chord plastification for gapped and overlapped connections and shear yielding (punching) for gapped connections only.

(a) For the limit state of chord plastification,

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

For the compression branch:

$$P_n \sin\theta = F_y t^2 [2.0 + 11.33 D_b/D] Q_g Q_f \tag{K2-6}$$

where D_b refers to the compression branch only, and

$$Q_g = \gamma^{0.2} \left[1 + \frac{0.024\gamma^{1.2}}{e^{\left(\frac{0.5g}{t} - 1.33\right)} + 1} \right] \tag{K2-7}$$

In gapped connections, g (measured along the crown of the chord neglecting weld dimensions) is positive. In overlapped connections, g is negative and equals q.

For the tension branch,

$$P_n \sin\theta = (P_n \sin\theta)_{\text{compression branch}} \tag{K2-8}$$

(b) For the limit state of shear yielding (punching) in gapped K-connections,

$$P_n = 0.6 F_y t \pi D_b [(1 + \sin\theta)/2\sin^2\theta] \tag{K2-9}$$

$$\phi = 0.95 \text{ (LRFD)} \qquad \Omega = 1.58 \text{ (ASD)}$$

3. Criteria for Rectangular HSS

The interaction of *stress* due to *chord member forces* and local branch connection forces shall be incorporated through the chord-stress interaction parameter Q_f.

When the chord is in tension,

$$Q_f = 1$$

When the chord is in compression in T-, Y-, and *cross-connections*,

$$Q_f = 1.3 - 0.4U/\beta \leq 1 \tag{K2-10}$$

When the chord is in compression in gapped K*-connections*,

$$Q_f = 1.3 - 0.4U/\beta_{eff} \leq 1 \tag{K2-11}$$

where U is the utilization ratio given by

$$U = |P_r/A_g F_c + M_r/SF_c| \tag{K2-12}$$

and

P_r = required axial strength in chord, kips (N). For gapped K-connections, P_r is to be determined on the side of the *joint* that has the higher compression stress (higher U).
M_r = required flexural strength in chord, kip-in. (N-mm)
A_g = chord gross area, in.^{2} (mm^2)
F_c = *available stress*, ksi (MPa)
S = chord elastic section modulus, in.^{3} (mm^3)

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

$P_r = P_u =$ required axial strength in chord, using *LRFD load combinations*, kips (N)
$M_r = M_u =$ required flexural strength in chord, using LRFD load combinations, kip-in. (N-mm)
$F_c = F_y$, ksi (MPa)

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

$P_r = P_a =$ required axial strength in chord, using *ASD load combinations*, kips, (N)
$M_r = M_a =$ required flexural strength in chord, using ASD load combinations, kip-in. (N-mm)
$F_c = 0.6F_y$, ksi, (MPa)

3a. Limits of Applicability

The criteria herein are applicable only when the *connection* configuration is within the following limits:

(1) *Joint eccentricity*: $-0.55H \leq e \leq 0.25H$, where H is the chord depth and e is positive away from the branches
(2) Branch angle: $\theta \geq 30°$
(3) Chord wall slenderness: ratio of overall wall width to thickness less than or equal to 35 for gapped K-connections and T-, Y- and cross-connections; less than or equal to 30 for overlapped K-connections
(4) Tension branch wall slenderness: ratio of overall wall width to thickness less than or equal to 35
(5) Compression branch wall slenderness: ratio of overall wall width to thickness less than or equal to $1.25(E/F_{yb})^{0.5}$ and also less than 35 for gapped K-connections and T-, Y- and cross-connections; less than or equal to $1.1(E/F_{yb})^{0.5}$ for overlapped K-connections
(6) Width ratio: ratio of overall wall width of branch to overall wall width of chord greater than or equal to 0.25 for T-, Y-, cross- and overlapped K-connections; greater than or equal to 0.35 for gapped K-connections
(7) Aspect ratio: $0.5 \leq$ ratio of depth to width ≤ 2.0
(8) Overlap: $25\% \leq O_v \leq 100\%$, where $O_v = (q/p) \times 100\%$. p is the projected length of the overlapping branch on the chord; q is the overlap length measured along the connecting face of the chord beneath the two branches. For overlap connections, the larger (or if equal width, the thicker) branch is a "thru member" connected directly to the chord
(9) Branch width ratio for *overlap connections*: ratio of overall wall width of overlapping branch to overall wall width of overlapped branch greater than or equal to 0.75
(10) Branch thickness ratio for overlap connections: thickness of overlapping branch to be less than or equal to the thickness of the overlapped branch

(11) Strength: $F_y \leq 52$ ksi (360 MPa) for chord and branches
(12) Ductility: $F_y/F_u \leq 0.8$
(13) Other limits apply for specific criteria

3b. Branches with Axial Loads in T-, Y- and Cross-Connections

For T-, Y-, and cross-connections, the *design strength* of the branch, ϕP_n, or the *allowable strength* of the branch, P_n/Ω, shall be the lowest value obtained according to the *limit states* of *chord wall plastification*, *shear yielding (punching)*, sidewall strength and *local yielding* due to *uneven load distribution*. In addition to the limits of applicability in Section K2.3a, β shall not be less than 0.25.

(a) For the limit state of chord wall plastification,

$$P_n \sin\theta = F_y t^2 [2\eta/(1-\beta) + 4/(1-\beta)^{0.5}] Q_f \tag{K2-13}$$

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

This limit state need not be checked when $\beta > 0.85$.

(b) For the limit state of shear yielding (punching),

$$P_n \sin\theta = 0.6 F_y t B [2\eta + 2\beta_{eop}] \tag{K2-14}$$

$$\phi = 0.95 \text{ (LRFD)} \qquad \Omega = 1.58 \text{ (ASD)}$$

In Equation K2-14, the effective outside punching parameter $\beta_{eop} = 5\beta/\gamma$ shall not exceed β.

This limit state need not be checked when $\beta > (1 - 1/\gamma)$, nor when $\beta < 0.85$ and $B/t \geq 10$.

(c) For the limit state of sidewall strength, the *available strength* for branches in tension shall be taken as the available strength for sidewall local yielding. For the limit state of sidewall strength, the available strength for branches in compression shall be taken as the lower of the strengths for sidewall local yielding and sidewall local crippling. For cross-connections with a branch angle less than 90 degrees, an additional check for chord sidewall shear failure must be made in accordance with Section G5.

This limit state need not be checked unless the chord member and branch member have the same width ($\beta = 1.0$).

(i) For the limit state of local yielding,

$$P_n \sin\theta = 2 F_y t [5k + N] \tag{K2-15}$$

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

where

k = outside corner radius of the *HSS*, which is permitted to be taken as $1.5t$ if unknown, in. (mm)

N = bearing length of the *load*, parallel to the axis of the HSS main member, $H_b/\sin\theta$, in. (mm)

(ii) For the limit state of sidewall local crippling, in T- and Y-connections,

$$P_n \sin\theta = 1.6t^2[1 + 3N/(H - 3t)](EF_y)^{0.5} Q_f \qquad \text{(K2-16)}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

(iii) For the limit state of sidewall local crippling in cross-connections,

$$P_n \sin\theta = [48t^3/(H - 3t)](EF_y)^{0.5} Q_f \qquad \text{(K2-17)}$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

(d) For the limit state of local yielding due to uneven load distribution,

$$P_n = F_{yb} t_b [2H_b + 2b_{eoi} - 4t_b] \qquad \text{(K2-18)}$$

$$\phi = 0.95 \text{ (LRFD)} \qquad \Omega = 1.58 \text{ (ASD)}$$

where

$$b_{eoi} = [10/(B/t)][F_y t/(F_{yb} t_b)]B_b \le B_b \qquad \text{(K2-19)}$$

This limit state need not be checked when $\beta < 0.85$.

3c. Branches with Axial Loads in Gapped K-Connections

For gapped K-connections, the design strength of the branch, ϕP_n, or the allowable strength of the branch, P_n/Ω, shall be the lowest value obtained according to the limit states of chord wall plastification, shear yielding (punching), *shear yielding* and local yielding due to uneven load distribution. In addition to the limits of applicability in Section K2.3a, the following limits shall apply:

(1) $B_b/B \ge 0.1 + \gamma/50$
(2) $\beta_{eff} \ge 0.35$
(3) $\zeta \ge 0.5(1 - \beta_{eff})$
(4) Gap: g greater than or equal to the sum of the branch wall thicknesses
(5) The smaller $B_b > 0.63$ times the larger B_b

(a) For the limit state of chord wall plastification,

$$P_n \sin\theta = F_y t^2 [9.8\beta_{eff}\gamma^{0.5}] Q_f \qquad \text{(K2-20)}$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

(b) For the limit state of shear yielding (punching),

$$P_n \sin\theta = 0.6F_y t B[2\eta + \beta + \beta_{eop}] \qquad \text{(K2-21)}$$

$$\phi = 0.95 \text{ (LRFD)} \qquad \Omega = 1.58 \text{ (ASD)}$$

In the above equation, the effective outside punching parameter $\beta_{eop} = 5\,\beta/\gamma$ shall not exceed β.

This limit state need only be checked if $B_b < (B - 2t)$ or the branch is not square.

(c) For the limit state of shear yielding of the chord in the gap, available strength shall be checked in accordance with Section G5. This limit state need only be checked if the chord is not square.

(d) For the limit state of local yielding due to uneven load distribution,

$$P_n = F_{yb}t_b[2H_b + B_b + b_{eoi} - 4t_b] \quad \text{(K2-22)}$$

$$\phi = 0.95 \text{ (LRFD)} \qquad \Omega = 1.58 \text{ (ASD)}$$

where

$$b_{eoi} = [10/(B/t)][F_y t/(F_{yb}t_b)]B_b \le B_b \quad \text{(K2-23)}$$

This limit state need only be checked if the branch is not square or $B/t < 15$.

3d. Branches with Axial Loads in Overlapped K-Connections

For overlapped K-connections, the design strength of the branch, ϕP_n, or the allowable strength of the branch, P_n/Ω, shall be determined from the limit state of local yielding due to uneven load distribution,

$$\phi = 0.95 \text{ (LRFD)} \qquad \Omega = 1.58 \text{ (ASD)}$$

For the overlapping branch, and for overlap $25\% \le O_v \le 50\%$ measured with respect to the overlapping branch,

$$P_n = F_{ybi}t_{bi}[(O_v/50)(2H_{bi} - 4t_{bi}) + b_{eoi} + b_{eov}] \quad \text{(K2-24)}$$

For the overlapping branch, and for overlap $50\% \le O_v < 80\%$ measured with respect to the overlapping branch,

$$P_n = F_{ybi}t_{bi}[2H_{bi} - 4t_{bi} + b_{eoi} + b_{eov}] \quad \text{(K2-25)}$$

For the overlapping branch, and for overlap $80\% \le O_v \le 100\%$ measured with respect to the overlapping branch,

$$P_n = F_{ybi}t_{bi}[2H_{bi} - 4t_{bi} + B_{bi} + b_{eov}] \quad \text{(K2-26)}$$

where

b_{eoi} is the *effective width* of the *branch face* welded to the chord,

$$b_{eoi} = [10/(B/t)][(F_y t)/(F_{ybi}t_{bi})]B_{bi} \le B_{bi} \quad \text{(K2-27)}$$

b_{eov} is the effective width of the branch face welded to the overlapped brace,

$$b_{eov} = [10/(B_{bj}/t_{bj})][(F_{ybj}t_{bj})/(F_{ybi}t_{bi})]\ B_{bi} \le B_{bi} \quad \text{(K2-28)}$$

B_{bi} = overall branch width of the overlapping branch, in. (mm)
B_{bj} = overall branch width of the overlapped branch, in. (mm)
F_{ybi} = *specified minimum yield stress* of the overlapping branch material, ksi (MPa)
F_{ybj} = specified minimum yield stress of the overlapped branch material, ksi (MPa)
H_{bi} = overall depth of the overlapping branch, in. (mm)
t_{bi} = thickness of the overlapping branch, in. (mm)
t_{bj} = thickness of the overlapped branch, in. (mm)

For the overlapped branch, P_n shall not exceed P_n of the overlapping branch, calculated using Equation K2-24, K2-25, or K2-26, as applicable, multiplied by the factor $(A_{bj}F_{ybj}/A_{bi}F_{ybi})$,

where

A_{bi} = cross-sectional area of the overlapping branch
A_{bj} = cross-sectional area of the overlapped branch

3e. Welds to Branches

The nonuniformity of load transfer along the line of weld, due to differences in relative flexibility of HSS walls in HSS-to-HSS connections, shall be considered in proportioning such welds. This can be considered by limiting the total effective weld length, L_e, of groove and *fillet welds* to rectangular HSS as follows:

(a) In T-, Y- and cross-connections,

for $\theta \leq 50$ degrees

$$L_e = \frac{2(H_b - 1.2t_b)}{\sin\theta} + (B_b - 1.2t_b) \tag{K2-29}$$

for $\theta \geq 60$ degrees

$$L_e = \frac{2(H_b - 1.2t_b)}{\sin\theta} \tag{K2-30}$$

Linear interpolation shall be used to determine L_e for values of θ between 50 and 60 degrees.

(b) In gapped K-connections, around each branch,

for $\theta \leq 50$ degrees

$$L_e = \frac{2(H_b - 1.2t_b)}{\sin\theta} + 2(B_b - 1.2t_b) \tag{K2-31}$$

for $\theta \geq 60$ degrees

$$L_e = \frac{2(H_b - 1.2t_b)}{\sin\theta} + (B_b - 1.2t_b) \tag{K2-32}$$

Linear interpolation shall be used to determine L_e for values of θ between 50 and 60 degrees.

In lieu of the above criteria in Equations K2-29 to K2-32, other rational criteria are permitted.

K3. HSS-TO-HSS MOMENT CONNECTIONS

HSS-to-HSS *moment connections* are defined as *connections* that consist of one or two *branch members* that are directly welded to a continuous chord that passes through the connection, with the branch or branches loaded by bending moments. A connection shall be classified

(a) As a *T-connection* when there is one branch and it is perpendicular to the chord and as a *Y-connection* when there is one branch but not perpendicular to the chord.

(b) As a *cross-connection* when there is a branch on each (opposite) side of the chord.

For the purposes of this Specification, the centerlines of the branch member(s) and the *chord member* shall lie in a common plane.

1. Definitions of Parameters

B = overall width of rectangular HSS main member, measured 90 degrees to the plane of the connection, in. (mm)
B_b = overall width of rectangular HSS branch member, measured 90 degrees to the plane of the connection, in. (mm)
D = outside diameter of round HSS main member, in. (mm)
D_b = outside diameter of round HSS branch member, in. (mm)
F_y = *specified minimum yield stress* of HSS main member, ksi (MPa)
F_{yb} = specified minimum yield stress of HSS branch member, ksi (MPa)
F_u = ultimate strength of HSS member, ksi (MPa)
H = overall height of rectangular HSS main member, measured in the plane of the connection, in. (mm)
H_b = overall height of rectangular HSS branch member, measured in the plane of the connection, in. (mm)
t = *design wall thickness* of HSS main member, in. (mm)
t_b = design wall thickness of HSS branch member, in. (mm)
β = the width ratio; the ratio of branch diameter to chord diameter = D_b/D for round HSS; the ratio of overall branch width to chord width = B_b/B for rectangular HSS
γ = the chord slenderness ratio; the ratio of one-half the diameter to the wall thickness = $D/2t$ for round HSS; the ratio of one-half the width to wall thickness = $B/2t$ for rectangular HSS
η = the *load* length parameter, applicable only to rectangular HSS; the ratio of the length of contact of the branch with the chord in the plane of the connection to the chord width = N/B, where $N = H_b/\sin\theta$
θ = acute angle between the branch and chord (degrees)

2. Criteria for Round HSS

The interaction of *stress* due to *chord member forces* and local branch *connection forces* shall be incorporated through the chord-stress interaction parameter Q_f.

When the chord is in tension,

$$Q_f = 1$$

When the chord is in compression,

$$Q_f = 1.0 - 0.3U(1 + U) \tag{K3-1}$$

where U is the utilization ratio given by

$$U = |P_r/A_gF_c + M_r/SF_c| \tag{K3-2}$$

and

P_r = required axial strength in chord, kips (N).
M_r = required flexural strength in chord, kip-in. (N-mm)
A_g = chord gross area, in.^{2} (mm^2)
F_c = *available stress*, ksi (MPa)
S = chord elastic section modulus, in.^{3} (mm^3)

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

$P_r = P_u$ = required axial strength in chord, using *LRFD load combinations*, kips (N)
$M_r = M_u$ = required flexural strength in chord, using LRFD load combinations, kip-in. (N-mm)
$F_c = F_y$, ksi (MPa)

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

$P_r = P_a$ = required axial strength in chord, using *ASD load combinations*, kips (N)
$M_r = M_a$ = required flexural strength in chord, using ASD load combinations, kip-in. (N-mm)
$F_c = 0.6F_y$, ksi (MPa)

2a. Limits of Applicability

The criteria herein are applicable only when the connection configuration is within the following limits of applicability:

(1) Branch angle: $\theta \geq 30°$
(2) Chord wall slenderness: ratio of diameter to wall thickness less than or equal to 50 for T- and *Y-connections*; less than or equal to 40 for *cross-connections*
(3) Tension branch wall slenderness: ratio of diameter to wall thickness less than or equal to 50
(4) Compression branch wall slenderness: ratio of diameter to wall thickness less than or equal to $0.05E/F_y$
(5) Width ratio: $0.2 < D_b/D \leq 1.0$
(6) Strength: $F_y \leq 52$ ksi (360 MPa) for chord and branches
(7) Ductility: $F_y/F_u \leq 0.8$

2b. Branches with In-Plane Bending Moments in T-, Y- and Cross-Connections

The *design strength*, ϕM_n, and the *allowable strength*, M_n/Ω, shall be the lowest value obtained according to the *limit states* of *chord plastification* and *shear yielding (punching)*.

(a) For the limit state of chord plastification,

$$M_n \sin\theta = 5.39 F_y t^2 \gamma^{0.5} \beta\, D_b\, Q_f \qquad \text{(K3-3)}$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

(b) For the limit state of shear yielding (punching),

$$M_n = 0.6 F_y t D_b^2 [(1 + 3\sin\theta)/4\sin^2\theta] \quad \text{(K3-4)}$$

$$\phi = 0.95 \text{ (LRFD)} \qquad \Omega = 1.58 \text{ (ASD)}$$

This limit state need not be checked when $\beta > (1 - 1/\gamma)$.

2c. Branches with Out-of-Plane Bending Moments in T-, Y- and Cross-Connections

The design strength, ϕM_n, and the allowable strength, M_n/Ω, shall be the lowest value obtained according to the limit states of chord plastification and shear yielding (punching).

(a) For the limit state of chord plastification,

$$M_n \sin\theta = F_y t^2 D_b [3.0/(1 - 0.81\beta)] Q_f \quad \text{(K3-5)}$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

(b) For the limit state of shear yielding (punching),

$$M_n = 0.6 F_y t D_b^2 [(3 + \sin\theta)/4\sin^2\theta] Q_f \quad \text{(K3-6)}$$

$$\phi = 0.95 \text{ (LRFD)} \qquad \Omega = 1.58 \text{ (ASD)}$$

This limit state need not be checked when $\beta > (1 - 1/\gamma)$.

2d. Branches with Combined Bending Moment and Axial Force in T-, Y- and Cross-Connections

Connections subject to branch axial load, branch in-plane bending moment, and branch out-of-plane bending moment, or any combination of these *load effects*, should satisfy the following.

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

$$(P_r/\phi P_n) + (M_{r\text{-}ip}/\phi M_{n\text{-}ip})^2 + (M_{r\text{-}op}/\phi M_{n\text{-}op}) \le 1.0 \quad \text{(K3-7)}$$

where

P_r = P_u = required axial strength in branch, using LRFD load combinations, kips (N)

ϕP_n = design strength obtained from Section K2.2b

$M_{r\text{-}ip}$ = required in-plane flexural strength in branch, using LRFD load combinations, kip-in. (N-mm)

$\phi M_{n\text{-}ip}$ = design strength obtained from Section K3.2b

$M_{r\text{-}op}$ = required out-of-plane flexural strength in branch, using LRFD load combinations, kip-in. (N-mm)

$\phi M_{n\text{-}op}$ = design strength obtained from Section K3.2c

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

$$(P_r/(P_n/\Omega)) + (M_{r\text{-}ip}/(M_{n\text{-}ip}/\Omega))^2 + (M_{r\text{-}op}/(M_{n\text{-}op}/\Omega)) \le 1.0 \quad \text{(K3-8)}$$

where

P_r = P_a = required axial strength in branch, using ASD load combinations, kips (N)

P_n/Ω = *allowable strength* obtained from Section K2.2b

$M_{r\text{-}ip}$ = required in-plane flexural strength in branch, using ASD load combinations, kip-in. (N-mm)

$M_{n\text{-}ip}/\Omega$ = allowable strength obtained from Section K3.2b

$M_{r\text{-}op}$ = required out-of-plane flexural strength in branch, using ASD load combinations, kip-in. (N-mm)

$M_{n\text{-}op}/\Omega$ = allowable strength obtained from Section K3.2c

3. Criteria for Rectangular HSS

The interaction of *stress* due to *chord member forces* and local branch *connection* forces shall be incorporated through the chord-stress interaction parameter Q_f.

When the chord is in tension,

$$Q_f = 1$$

When the chord is in compression,

$$Q_f = (1.3 - 0.4U/\beta) \leq 1 \quad \text{(K3-9)}$$

where U is the utilization ratio given by

$$U = |P_r/A_g F_c + M_r/SF_c| \quad \text{(K3-10)}$$

and

P_r = required axial strength in chord, kips (N)

M_r = required flexural strength in chord, kip-in. (N-mm)

A_g = chord gross area, in.^{2} (mm^2)

F_c = *available stress*, ksi, (MPa)

S = chord elastic section modulus, in.^{3} (mm^3)

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

P_r = P_u = required axial strength in chord, using *LRFD load combinations*, kips, (N)

M_r = M_u = required flexural strength in chord, using LRFD load combinations, kip-in. (N-mm)

F_c = F_y, ksi, (MPa)

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

P_r = P_a = required axial strength in chord, using *ASD load combinations*, kips, (N)

M_r = M_a = required flexural strength in chord, using ASD load combinations, kip-in. (N-mm)

F_c = $0.6F_y$, ksi, (MPa)

3a. Limits of Applicability

The criteria herein are applicable only when the *connection* configuration is within the following limits:

(1) Branch angle is approximately 90°
(2) Chord wall slenderness: ratio of overall wall width to thickness less than or equal to 35
(3) Tension branch wall slenderness: ratio of overall wall width to thickness less than or equal to 35
(4) Compression branch wall slenderness: ratio of overall wall width to thickness less than or equal to $1.25(E/F_{yb})^{0.5}$ and also less than 35
(5) Width ratio: ratio of overall wall width of branch to overall wall width of chord greater than or equal to 0.25
(6) Aspect ratio: $0.5 \leq$ ratio of depth to width ≤ 2.0
(7) Strength: $F_y \leq 52$ ksi (360 MPa) for chord and branches
(8) Ductility: $F_y/F_u \leq 0.8$
(9) Other limits apply for specific criteria

3b. Branches with In-Plane Bending Moments in T- and Cross-Connections

The *design strength*, ϕM_n, and the *allowable strength*, M_n/Ω, shall be the lowest value obtained according to the *limit states* of *chord wall plastification*, sidewall *local yielding* and local yielding due to *uneven load distribution*.

(a) For the limit state of chord wall plastification,

$$M_n = F_y t^2 H_b[(1/2\eta) + 2/(1-\beta)^{0.5} + \eta/(1-\beta)]Q_f \quad \text{(K3-11)}$$

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

This limit state need not be checked when $\beta > 0.85$.

(b) For the limit state of sidewall local yielding,

$$M_n = 0.5F_y{}^* t(H_b + 5t)^2 \quad \text{(K3-12)}$$

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

where

$F_y{}^* = F_y$ for *T-connections*
$F_y{}^* = 0.8F_y$ for *cross-connections*

This limit state need not be checked when $\beta < 0.85$.

(c) For the limit state of local yielding due to uneven load distribution,

$$M_n = F_{yb}[Z_b - (1 - b_{eoi}/B_b)B_b H_b t_b] \quad \text{(K3-13)}$$

$$\phi = 0.95 \text{ (LRFD)} \qquad \Omega = 1.58 \text{ (ASD)}$$

where

$b_{eoi} = [10/(B/t)][F_y t/(F_{yb} t_b)]B_b \leq B_b$ (K3-14)

Z_b = branch plastic section modulus about the axis of bending, in.^{3}(mm^3)

This limit state need not be checked when $\beta < 0.85$.

3c. Branches with Out-of-Plane Bending Moments in T- and Cross-Connections

The design strength, ϕM_n, and the allowable strength, M_n/Ω, shall be the lowest value obtained according to the limit states of chord wall plastification, sidewall local yielding, local yielding due to uneven load distribution and chord *distortional failure.*

(a) For the limit state of chord wall plastification,

$$M_n = F_y t^2[0.5H_b(1+\beta)/(1-\beta) + [2BB_b(1+\beta)/(1-\beta)]^{0.5}]Q_f \quad \text{(K3-15)}$$

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

This limit state need not be checked when $\beta > 0.85$.

(b) For the limit state of sidewall local yielding,

$$M_n = F_y{}^* t(B-t)(H_b + 5t) \quad \text{(K3-16)}$$

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

where

$F_y{}^* = F_y$ for T-connections

$F_y{}^* = 0.8F_y$ for cross-connections

This limit state need not be checked when $\beta < 0.85$.

(c) For the limit state of local yielding due to uneven load distribution,

$$M_n = F_{yb}[Z_b - 0.5(1 - b_{eoi}/B_b)^2 B_b{}^2 t_b] \quad \text{(K3-17)}$$

$$\phi = 0.95 \text{ (LRFD)} \qquad \Omega = 1.58 \text{ (ASD)}$$

where

$b_{eoi} = [10/(B/t)][F_y t/(F_{yb} t_b)]B_b \leq B_b$ (K3-18)

Z_b = branch plastic section modulus about the axis of bending, in.^{3} (mm^3)

This limit state need not be checked when $\beta < 0.85$.

(d) For the limit state of chord distortional failure,

$$M_n = 2F_y t[H_b t + [BHt(B+H)]^{0.5}] \quad \text{(K3-19)}$$

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

This limit state need not be checked for cross-connections or for T-connections if chord distortional failure is prevented by other means.

3d. Branches with Combined Bending Moment and Axial Force in T- and Cross-Connections

Connections subject to branch axial *load*, branch in-plane bending moment, and branch out-of-plane bending moment, or any combination of these *load effects*, should satisfy

For design according to Section B3.3 (LRFD)

$$(P_r/\phi P_n) + (M_{r\text{-}ip}/\phi M_{n\text{-}ip}) + (M_{r\text{-}op}/\phi M_{n\text{-}op}) \leq 1.0 \qquad \text{(K3-20)}$$

where

P_r = P_u = required axial strength in branch, using LRFD load combinations, kips (N)

ϕP_n = design strength obtained from Section K2.3b

$M_{r\text{-}ip}$ = required in-plane flexural strength in branch, using LRFD load combinations, kip-in. (N-mm)

$\phi M_{n\text{-}ip}$ = design strength obtained from Section K3.3b

$M_{r\text{-}op}$ = required out-of-plane flexural strength in branch, using LRFD load combinations, kip-in. (N-mm)

$\phi M_{n\text{-}op}$ = design strength obtained from Section K3.3c

For design according to Section B3.4 (ASD)

$$(P_r/(P_n/\Omega)) + (M_{r\text{-}ip}/(M_{n\text{-}ip}/\Omega)) + (M_{r\text{-}op}/(M_{n\text{-}op}/\Omega)) \leq 1.0 \qquad \text{(K3-21)}$$

where

P_r = P_a = required axial strength in branch, using ASD load combinations, kips (N)

P_n/Ω = allowable strength obtained from Section K2.3b

$M_{r\text{-}ip}$ = required in-plane flexural strength in branch, using ASD load combinations, kip-in. (N-mm)

$M_{n\text{-}ip}/\Omega$ = allowable strength obtained from Section K3.3b

$M_{r\text{-}op}$ = required out-of-plane flexural strength in branch, using ASD load combinations, kip-in. (N-mm)

$M_{n\text{-}op}/\Omega$ = allowable strength obtained from Section K3.3c

CHAPTER L

DESIGN FOR SERVICEABILITY

This chapter addresses *serviceability* performance design requirements.

The chapter is organized as follows:

L1. General Provisions
L2. Camber
L3. Deflections
L4. Drift
L5. Vibration
L6. Wind-Induced Motion
L7. Expansion and Contraction
L8. Connection Slip

L1. GENERAL PROVISIONS

Serviceability is a state in which the function of a building, its appearance, maintainability, durability, and comfort of its occupants are preserved under normal usage. Limiting values of structural behavior for serviceability (for example, maximum deflections, accelerations) shall be chosen with due regard to the intended function of the structure. Serviceability shall be evaluated using appropriate *load combinations* for the serviceability *limit states* identified.

User Note: Additional information on serviceability limit states, service loads and appropriate load combinations for serviceability requirements can be found in ASCE 7, Appendix B and its Commentary. The performance requirements for serviceability in this chapter are consistent with those requirements. Service loads, as stipulated herein, are those that act on the structure at an arbitrary point in time. That is, the appropriate load combinations are often less severe than those in ASCE 7, Section 2.4, where the LRFD load combinations are given.

L2. CAMBER

Where *camber* is used to achieve proper position and location of the structure, the magnitude, direction and location of camber shall be specified in the structural drawings.

User Note: Camber recommendations are provided in the *Code of Standard Practice for Steel Buildings and Bridges*.

L3. DEFLECTIONS

Deflections in structural members and structural systems under appropriate *service load combinations* shall not impair the *serviceability* of the structure.

User Note: Conditions to be considered include levelness of floors, alignment of structural members, integrity of building finishes, and other factors that affect the normal usage and function of the structure. For *composite* members, the additional deflections due to the shrinkage and creep of the concrete should be considered.

L4. DRIFT

Drift of a structure shall be evaluated under *service loads* to provide for *serviceability* of the structure, including the integrity of interior partitions and exterior *cladding*. *Drift* under strength *load combinations* shall not cause collision with adjacent structures or exceed the limiting values of such drifts that may be specified by the *applicable building code*.

L5. VIBRATION

The effect of vibration on the comfort of the occupants and the function of the structure shall be considered. The sources of vibration to be considered include pedestrian loading, vibrating machinery and others identified for the structure.

L6. WIND-INDUCED MOTION

The effect of wind-induced motion of buildings on the comfort of occupants shall be considered.

L7. EXPANSION AND CONTRACTION

The effects of thermal expansion and contraction of a building shall be considered. Damage to building *cladding* can cause water penetration and may lead to corrosion.

L8. CONNECTION SLIP

The effects of *connection* slip shall be included in the design where slip at bolted connections may cause deformations that impair the *serviceability* of the structure. Where appropriate, the connection shall be designed to preclude slip. For the design of slip-critical connections see Sections J3.8 and J3.9.

User Note: For more information on connection slip, refer to the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts*.

CHAPTER M

FABRICATION, ERECTION AND QUALITY CONTROL

This chapter addresses requirements for shop drawings, fabrication, shop painting, erection and *quality control*.

The chapter is organized as follows:

M1. SHOP AND ERECTION DRAWINGS

Shop drawings shall be prepared in advance of fabrication and give complete information necessary for the fabrication of the component parts of the structure, including the location, type and size of welds and bolts. Erection drawings shall be prepared in advance of erection and give information necessary for erection of the structure. Shop and erection drawings shall clearly distinguish between shop and field welds and bolts and shall clearly identify pretensioned and slip-critical high-strength bolted *connections*. Shop and erection drawings shall be made with due regard to speed and economy in fabrication and erection.

M2. FABRICATION

1. Cambering, Curving and Straightening

Local application of heat or mechanical means is permitted to be used to introduce or correct *camber*, curvature and straightness. The temperature of heated areas, as measured by approved methods, shall not exceed 1,100 °F (593 °C) for A514/A514M and A852/A852M steel nor 1,200 °F (649 °C) for other steels.

2. Thermal Cutting

Thermally cut edges shall meet the requirements of AWS D1.1, Sections 5.15.1.2, 5.15.4.3 and 5.15.4.4 with the exception that thermally cut free edges that will be subject to calculated static tensile *stress* shall be free of round-bottom *gouges* greater than 3/16 in. (5 mm) deep and sharp V-shaped notches. *Gouges* deeper than 3/16 in. (5 mm) and notches shall be removed by grinding or repaired by welding.

Reentrant corners, except reentrant corners of *beam copes* and weld access holes, shall meet the requirements of AWS D1.1, Section A5.16. If another specified contour is required it must be shown on the contract documents.

Beam copes and weld access holes shall meet the geometrical requirements of Section J1.6. Beam copes and weld access holes in shapes that are to be galvanized shall be ground. For shapes with a flange thickness not exceeding 2 in. (50 mm) the roughness of *thermally cut* surfaces of copes shall be no greater than a surface roughness value of 2,000 μin. (50 μm) as defined in ASME B46.1 Surface Texture (*Surface Roughness, Waviness, and Lay*). For beam copes and weld access holes in which the curved part of the access hole is thermally cut in ASTM A6/A6M hot-rolled shapes with a flange thickness exceeding 2 in. (50 mm) and welded built-up shapes with material thickness greater than 2 in. (50 mm), a preheat temperature of not less than 150 °F (66 °C) shall be applied prior to thermal cutting. The thermally cut surface of access holes in ASTM A6/A6M hot-rolled shapes with a flange thickness exceeding 2 in. (50 mm) and built-up shapes with a material thickness greater than 2 in. (50 mm) shall be ground and inspected for cracks using magnetic particle inspection in accordance with ASTM E709. Any crack is unacceptable regardless of size or location.

User Note: The AWS Surface Roughness Guide for Oxygen Cutting (AWS C4.1-77) sample 3 may be used as a guide for evaluating the surface roughness of *copes* in shapes with flanges not exceeding 2 in. (50 mm) thick.

3. Planing of Edges

Planing or finishing of sheared or *thermally cut* edges of plates or shapes is not required unless specifically called for in the contract documents or included in a stipulated edge preparation for welding.

4. Welded Construction

The technique of welding, the workmanship, appearance and quality of welds, and the methods used in correcting nonconforming work shall be in accordance with AWS D1.1 except as modified in Section J2.

5. Bolted Construction

Parts of bolted members shall be pinned or bolted and rigidly held together during assembly. Use of a *drift* pin in bolt holes during assembly shall not distort the metal or enlarge the holes. Poor matching of holes shall be cause for rejection.

Bolt holes shall comply with the provisions of the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, Section 3.3 except that *thermally cut* holes shall be permitted with a surface roughness profile not exceeding 1,000 μin. (25 μm) as defined in ASME B46.1. *Gouges* shall not exceed a depth of $^1/_{16}$ in. (2 mm).

Fully inserted finger *shims*, with a total thickness of not more than 1/4 in. (6 mm) within a *joint*, are permitted in *joints* without changing the strength (based upon hole type) for the design of *connections*. The orientation of such *shims* is independent of the direction of application of the *load*.

The use of high-strength bolts shall conform to the requirements of the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, except as modified in Section J3.

6. Compression Joints

Compression *joints* that depend on contact bearing as part of the *splice* strength shall have the bearing surfaces of individual fabricated pieces prepared by milling, sawing, or other suitable means.

7. Dimensional Tolerances

Dimensional tolerances shall be in accordance with the AISC *Code of Standard Practice for Steel Buildings and Bridges*.

8. Finish of Column Bases

Column bases and base plates shall be finished in accordance with the following requirements:

(1) Steel bearing plates 2 in. (50 mm) or less in thickness are permitted without milling, provided a satisfactory contact bearing is obtained. Steel bearing plates over 2 in. (50 mm) but not over 4 in. (100 mm) in thickness are permitted to be straightened by pressing or, if presses are not available, by milling for bearing surfaces (except as noted in subparagraphs 2 and 3 of this section), to obtain a satisfactory contact bearing. Steel bearing plates over 4 in. (100 mm) in thickness shall be milled for bearing surfaces (except as noted in subparagraphs 2 and 3 of this section).

(2) Bottom surfaces of bearing plates and *column* bases that are grouted to ensure full bearing contact on foundations need not be milled.

(3) Top surfaces of bearing plates need not be milled when complete-*joint*-penetration *groove welds* are provided between the *column* and the bearing plate.

9. Holes for Anchor Rods

Holes for anchor rods shall be permitted to be *thermally cut* in accordance with the provisions of Section M2.2.

10. Drain Holes

When water can collect inside *HSS* or box members, either during construction or during service, the member shall be sealed, provided with a drain hole at the base, or protected by other suitable means.

11. Requirements for Galvanized Members

Members and parts to be galvanized shall be designed, detailed and fabricated to provide for flow and drainage of pickling fluids and zinc and to prevent pressure build-up in enclosed parts.

> **User Note:** See *The Design of Products to be Hot-Dip Galvanized After Fabrication*, American Galvanizer's Association, and ASTM A123, A153, A384 and A780 for useful information on design and detailing of galvanized members.

M3. SHOP PAINTING

1. General Requirements

Shop painting and surface preparation shall be in accordance with the provisions of the AISC *Code of Standard Practice for Steel Buildings and Bridges*.

Shop paint is not required unless specified by the contract documents.

2. Inaccessible Surfaces

Except for contact surfaces, surfaces inaccessible after shop assembly shall be cleaned and painted prior to assembly, if required by the design documents.

3. Contact Surfaces

Paint is permitted in *bearing-type connection*s. For *slip-critical connection*s, the *faying surface* requirements shall be in accordance with the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, Section 3.2.2(b).

4. Finished Surfaces

Machine-finished surfaces shall be protected against corrosion by a rust inhibitive coating that can be removed prior to erection, or which has characteristics that make removal prior to erection unnecessary.

5. Surfaces Adjacent to Field Welds

Unless otherwise specified in the design documents, surfaces within 2 in. (50 mm) of any field weld location shall be free of materials that would prevent proper welding or produce objectionable fumes during welding.

M4. ERECTION

1. Alignment of Column Bases

Column bases shall be set level and to correct elevation with full bearing on concrete or masonry.

2. Bracing

The frame of steel skeleton buildings shall be carried up true and plumb within the limits defined in the AISC *Code of Standard Practice for Steel Buildings and*

Bridges. Temporary bracing shall be provided, in accordance with the requirements of the *Code of Standard Practice for Steel Buildings and Bridges*, wherever necessary to support the loads to which the structure may be subjected, including equipment and the operation of same. Such bracing shall be left in place as long as required for safety.

3. Alignment

No permanent bolting or welding shall be performed until the adjacent affected portions of the structure have been properly aligned.

4. Fit of Column Compression Joints and Base Plates

Lack of contact bearing not exceeding a gap of $^1/_{16}$ in. (2 mm), regardless of the type of *splice* used (*partial-joint-penetration groove welded* or bolted), is permitted. If the gap exceeds $^1/_{16}$ in. (2 mm), but is less than $^1/_4$ in. (6 mm), and if an engineering investigation shows that sufficient contact area does not exist, the gap shall be packed out with nontapered steel *shim*s. Shims need not be other than mild steel, regardless of the grade of the main material.

5. Field Welding

Shop paint on surfaces adjacent to *joint*s to be field welded shall be wire brushed if necessary to assure weld quality.

Field welding of attachments to installed embedments in contact with concrete shall be done in such a manner as to avoid excessive thermal expansion of the embedment which could result in spalling or cracking of the concrete or excessive *stress* in the embedment anchors.

6. Field Painting

Responsibility for touch-up painting, cleaning and field painting shall be allocated in accordance with accepted local practices, and this allocation shall be set forth explicitly in the design documents.

7. Field Connections

As erection progresses, the structure shall be securely bolted or welded to support the dead, wind and erection *load*s.

M5. QUALITY CONTROL

The fabricator shall provide *quality control* procedures to the extent that the fabricator deems necessary to assure that the work is performed in accordance with this Specification. In addition to the fabricator's quality control procedures, material and workmanship at all times may be subject to inspection by qualified inspectors representing the purchaser. If such inspection by representatives of the purchaser will be required, it shall be so stated in the design documents.

1. Cooperation

As far as possible, the inspection by representatives of the purchaser shall be made at the fabricator's plant. The fabricator shall cooperate with the inspector, permitting access for inspection to all places where work is being done. The purchaser's inspector shall schedule this work for minimum interruption to the work of the fabricator.

2. Rejections

Material or workmanship not in conformance with the provisions of this Specification may be rejected at any time during the progress of the work.

The fabricator shall receive copies of all reports furnished to the purchaser by the inspection agency.

3. Inspection of Welding

The inspection of welding shall be performed in accordance with the provisions of AWS D1.1 except as modified in Section J2.

When visual inspection is required to be performed by AWS certified welding inspectors, it shall be so specified in the design documents.

When nondestructive testing is required, the process, extent and standards of acceptance shall be clearly defined in the design documents.

4. Inspection of Slip-Critical High-Strength Bolted Connections

The inspection of slip-critical high-strength bolted *connection*s shall be in accordance with the provisions of the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts*.

5. Identification of Steel

The fabricator shall be able to demonstrate by a written procedure and by actual practice a method of material identification, visible at least through the "fit-up" operation, for the main structural elements of each shipping piece.

APPENDIX 1

INELASTIC ANALYSIS AND DESIGN

Design by *inelastic analysis* is subject to the supplementary provisions of this appendix.

The appendix is organized as follows:

1.1. GENERAL PROVISIONS

Inelastic analysis is permitted for design according to the provisions of Section B3.3 (LRFD). Inelastic analysis is not permitted for design according to the provisions of Section B3.4 (ASD) except as provided in Section 1.3.

1.2. MATERIALS

Members undergoing plastic hinging shall have a *specified minimum yield stress* not exceeding 65 ksi (450 MPa).

1.3. MOMENT REDISTRIBUTION

*Beam*s and girders composed of *compact sections* as defined in Section B4 and satisfying the *unbraced length* requirements of Section 1.7, including *composite* members, may be proportioned for nine-tenths of the negative moments at points of support, produced by the *gravity loading* computed by an *elastic analysis*, provided that the maximum positive moment is increased by one-tenth of the average negative moments. This reduction is not permitted for moments produced by loading on cantilevers and for design according to Sections 1.4 through 1.8 of this appendix.

If the negative moment is resisted by a *column* rigidly framed to the beam or girder, the one-tenth reduction may be used in proportioning the column for combined axial *force* and flexure, provided that the axial force does not exceed $0.15\phi_c F_y A_g$ for LRFD or $0.15 F_y A_g/\Omega_c$ for ASD,

where

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

F_y = *specified minimum yield stress* of the compression flange, ksi (MPa)

ϕ_c = *resistance factor* for compression = 0.90
Ω_c = *safety factor* for compression = 1.67

1.4. LOCAL BUCKLING

Flanges and webs of members subject to plastic hinging in combined flexure and axial compression shall be compact with width-thickness ratios less than or equal to the limiting λ_p defined in Table B4.1 or as modified as follows:

(a) For webs of doubly symmetric wide flange members and rectangular *HSS* in combined flexure and compression

(i) For $P_u/\phi_b P_y \le 0.125$

$$h/t_w \le 3.76\sqrt{\frac{E}{F_y}}\left(1 - \frac{2.75P_u}{\phi_b P_y}\right) \tag{A-1-1}$$

(ii) For $P_u/\phi_b P_y > 0.125$

$$h/t_w \le 1.12\sqrt{\frac{E}{F_y}}\left(2.33 - \frac{P_u}{\phi_b P_y}\right) \ge 1.49\sqrt{\frac{E}{F_y}} \tag{A-1-2}$$

where

E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)
F_y = specified minimum yield stress of the type of steel being used, ksi (MPa)
h = as defined in Section B4.2, in. (mm)
P_u = required axial strength in compression, kips (N)
P_y = member yield strength, kips (N)
t_w = web thickness, in. (mm)
ϕ_b = resistance factor for flexure = 0.90

(b) For flanges of rectangular box and hollow structural sections of uniform thickness subject to bending or compression, flange *cover plates*, and *diaphragm plates* between lines of *fasteners* or welds

$$b/t \le 0.94\sqrt{E/F_y} \tag{A-1-3}$$

where

b = as defined in Section B4.2, in. (mm)
t = as defined in Section B4.2, in. (mm)

(c) For circular hollow sections in flexure

$$D/t \le 0.045E/F_y \tag{A-1-4}$$

where

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

1.5. STABILITY AND SECOND-ORDER EFFECTS

Continuous *beams* not subjected to axial *loads* and that do not contribute to lateral *stability* of framed structures may be designed based on a *first-order inelastic analysis* or a plastic *mechanism* analysis.

Braced frames and *moment frames* may be designed based on a *first-order inelastic analysis* or a plastic *mechanism* analysis provided that *stability* and *second-order effects* are taken into account.

Structures may be designed on the basis of a second-order *inelastic analysis*. For *beam-columns*, *connections* and connected members, the *required strengths* shall be determined from a second-order inelastic analysis, where equilibrium is satisfied on the deformed geometry, taking into account the change in *stiffness* due to yielding.

1. Braced Frames

In *braced frames* designed on the basis of *inelastic analysis*, braces shall be designed to remain elastic under the *design loads*. The required axial strength for *columns* and compression braces shall not exceed $\phi_c\ (0.85F_yA_g)$,

where

$$\phi_c = 0.90\,(\text{LRFD})$$

2. Moment Frames

In *moment frames* designed on the basis of *inelastic analysis*, the required axial strength of columns shall not exceed $\phi_c(0.75F_yA_g)$,

where

$$\phi_c = 0.90\,(\text{LRFD})$$

1.6. COLUMNS AND OTHER COMPRESSION MEMBERS

In addition to the limits set in Sections 1.5.1 and 1.5.2, the required axial strength of *columns* designed on the basis of *inelastic analysis* shall not exceed the design strength, $\phi_c P_n$, determined according to the provisions of Section E3.

Design by inelastic analysis is permitted if the column slenderness ratio, L/r, does not exceed $4.71\sqrt{E/F_y}$,

where

L = laterally unbraced length of a member, in. (mm)
r = governing radius of gyration, in. (mm)

User Note: A well-proportioned member will not be expected to reach this limit.

1.7. BEAMS AND OTHER FLEXURAL MEMBERS

The required moment strength, M_u, of *beams* designed on the basis of *inelastic analysis* shall not exceed the *design strength*, ϕM_n, where

$$M_n = M_p = F_y Z < 1.6 F_y S \tag{A-1-6}$$

$$\phi = 0.90\,(\text{LRFD})$$

Design by inelastic analysis is permitted for members that are compact as defined in Section B4 and as modified in Section 1.4.

The laterally *unbraced length*, L_b, of the compression flange adjacent to *plastic hinge* locations shall not exceed L_{pd}, determined as follows.

(a) For doubly symmetric and singly symmetric I-shaped members with the compression flange equal to or larger than the tension flange loaded in the plane of the web:

$$L_{pd} = \left[0.12 + 0.076\left(\frac{M_1}{M_2}\right)\right]\left(\frac{E}{F_y}\right) r_y \tag{A-1-7}$$

where

M_1 = smaller moment at end of unbraced length of beam, kip-in. (N-mm)
M_2 = larger moment at end of unbraced length of beam, kip-in. (N-mm)
r_y = radius of gyration about minor axis, in. (mm)

(M_1/M_2) is positive when moments cause *reverse curvature* and negative for *single curvature*.

(b) For solid rectangular bars and symmetric box beams:

$$L_{pd} = \left[0.17 + 0.10\left(\frac{M_1}{M_2}\right)\right]\left(\frac{E}{F_y}\right) r_y \geq 0.10\left(\frac{E}{F_y}\right) r_y \tag{A-1-8}$$

There is no limit on L_b for members with circular or square cross sections or for any beam bent about its minor axis.

1.8. MEMBERS UNDER COMBINED FORCES

When inelastic analysis is used for symmetric members subject to bending and axial force, the provisions in Section H1 apply.

Inelastic analysis is not permitted for members subject to torsion and combined torsion, flexure, shear and/or axial force.

1.9. CONNECTIONS

Connections adjacent to plastic hinging regions of connected members shall be designed with sufficient strength and ductility to sustain the *forces* and deformations imposed under the required *loads*.

APPENDIX 2

DESIGN FOR PONDING

This appendix provides methods for determining whether a roof system has adequate strength and stiffness to resist ponding.

The appendix is organized as follows:

2.1. Simplified Design for Ponding
2.2. Improved Design for Ponding

2.1. SIMPLIFIED DESIGN FOR PONDING

The roof system shall be considered stable for *ponding* and no further investigation is needed if both of the following two conditions are met:

$$C_p + 0.9C_s \le 0.25 \tag{A-2-1}$$

$$I_d \ge 25(S^4)10^{-6} \tag{A-2-2}$$

$$I_d \ge 3940\, S^4 \text{ (S.I.)} \tag{A-2-2M}$$

where

$$C_p = \frac{32 L_s L_p^4}{10^7 I_p}$$

$$C_p = \frac{504 L_s L_p^4}{I_p} \text{ (S.I.)}$$

$$C_s = \frac{32 S L_s^4}{10^7 I_s}$$

$$C_s = \frac{504 S L_s^4}{I_s} \text{ (S.I.)}$$

L_p = *column* spacing in direction of girder (length of primary members), ft (m)

L_s = column spacing perpendicular to direction of girder (length of secondary members), ft (m)

S = spacing of secondary members, ft (m)

I_p = moment of inertia of primary members, in.^{4} (mm^4)

I_s = moment of inertia of secondary members, in.^{4} (mm^4)

I_d = moment of inertia of the steel deck supported on secondary members, in.^{4}per ft (mm^4 per m)

For trusses and steel joists, the moment of inertia I_s shall be decreased 15 percent when used in the above equation. A steel deck shall be considered a secondary member when it is directly supported by the primary members.

2.2. IMPROVED DESIGN FOR PONDING

The provisions given below are permitted to be used when a more exact determination of framing *stiffness* is needed than that given in Section 2.1.

For primary members, the stress index shall be

$$U_p = \left(\frac{0.8F_y - f_o}{f_o}\right)_p \tag{A-2-3}$$

For secondary members, the stress index shall be

$$U_s = \left(\frac{0.8F_y - f_o}{f_o}\right)_s \tag{A-2-4}$$

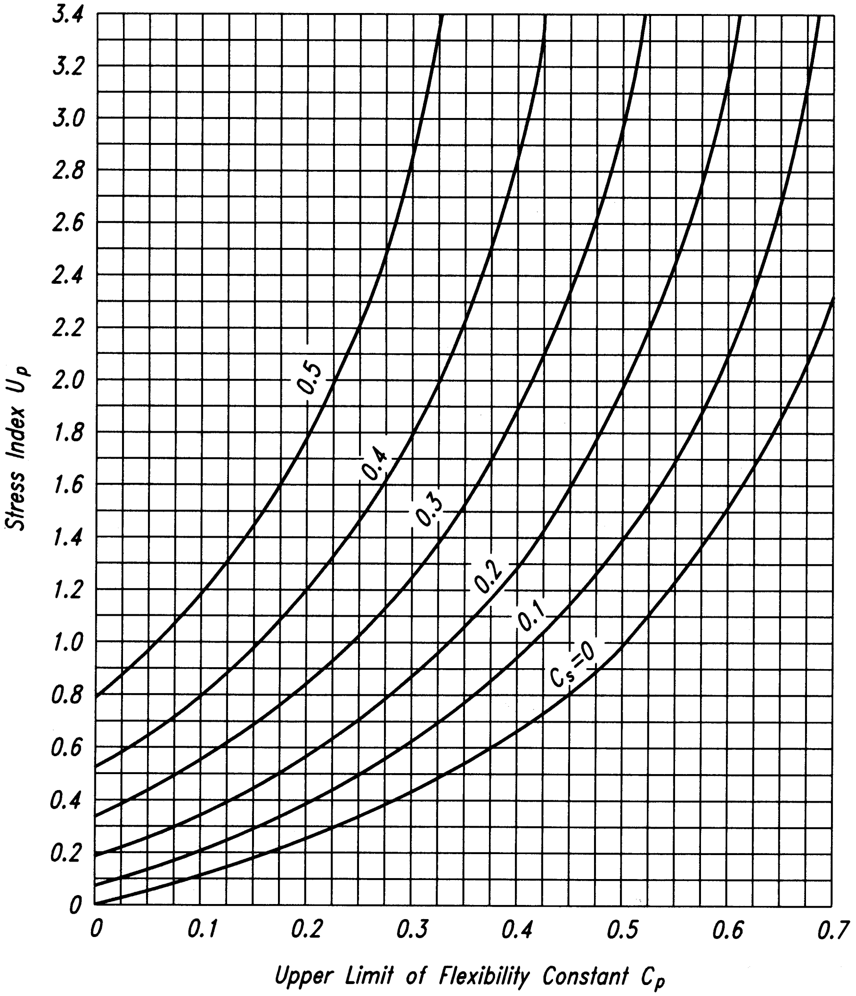


Fig. A-2-1. Limiting flexibility coefficient for the primary systems.

where

f_o = *stress* due to the load combination $(D + R)$
D = nominal dead *load*
R = *nominal load* due to rainwater or snow, exclusive of the *ponding* contribution, ksi (MPa)

For roof framing consisting of primary and secondary members, the combined stiffness shall be evaluated as follows: enter Figure A-2-1 at the level of the computed stress index U_p determined for the primary beam; move horizontally to the computed C_s value of the secondary beams and then downward to the abscissa scale. The combined stiffness of the primary and secondary framing is sufficient to prevent ponding if the flexibility constant read from this latter scale is more than the value of C_p computed for the given primary member; if not, a stiffer primary or secondary beam, or combination of both, is required.

A similar procedure must be followed using Figure A-2-2.

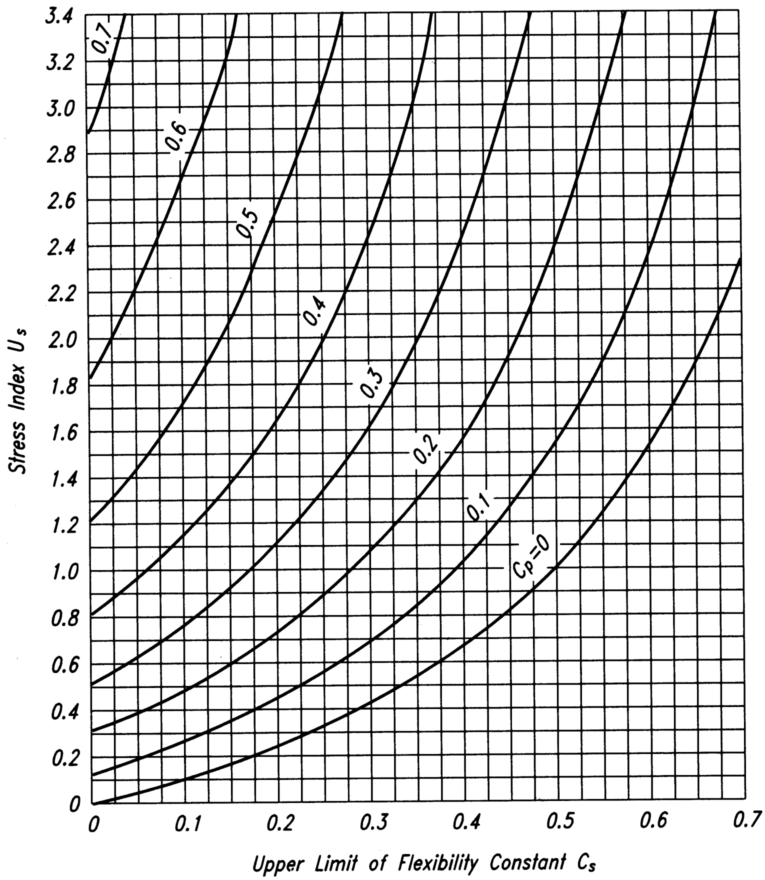


Fig. A-2-2. Limiting flexibility coefficient for the secondary systems.

For roof framing consisting of a series of equally spaced wall-bearing beams, the stiffness shall be evaluated as follows. The beams are considered as secondary members supported on an infinitely stiff primary member. For this case, enter Figure A-2-2 with the computed *stress* index U_s. The limiting value of C_s is determined by the intercept of a horizontal line representing the U_s value and the curve for $C_p = 0$.

User Note: The *ponding* deflection contributed by a metal deck is usually such a small part of the total ponding deflection of a roof panel that it is sufficient merely to limit its moment of inertia (per foot (meter) of width normal to its span) to $0.000025l^4$ in.^{4}/ft ($3940l^4$ mm^4/m).

For roof framing consisting of metal deck spanning between beams supported on columns, the stiffness shall be evaluated as follows. Employ Figure A-2-1 or A-2-2 using as C_s the flexibility constant for a 1 ft (1 m) width of the roof deck ($S = 1.0$).

APPENDIX 3

DESIGN FOR FATIGUE

This appendix applies to members and *connections* subject to high cycle loading within the elastic range of stresses of frequency and magnitude sufficient to initiate cracking and progressive failure, which defines the *limit state* of *fatigue*.

The appendix is organized as follows:

3.1. General
3.2. Calculation of Maximum Stresses and Stress Ranges
3.3. Design Stress Range
3.4. Bolts and Threaded Parts
3.5. Special Fabrication and Erection Requirements

3.1. GENERAL

The provisions of this Appendix apply to stresses calculated on the basis of *service loads*. The maximum permitted *stress* due to unfactored *loads* is $0.66F_y$.

Stress range is defined as the magnitude of the change in stress due to the application or removal of the service live load. In the case of a stress reversal, the stress range shall be computed as the numerical sum of maximum repeated tensile and compressive stresses or the numerical sum of maximum shearing stresses of opposite direction at the point of probable crack initiation.

In the case of complete-joint-penetration butt welds, the maximum *design stress range* calculated by Equation A-3-1 applies only to welds with internal soundness meeting the acceptance requirements of Section 6.12.2 or 6.13.2 of AWS D1.1.

No evaluation of *fatigue* resistance is required if the live load stress range is less than the threshold stress range, F_{TH}. See Table A-3.1.

No evaluation of fatigue resistance is required if the number of cycles of application of live load is less than 20,000.

The cyclic load resistance determined by the provisions of this Appendix is applicable to structures with suitable corrosion protection or subject only to mildly corrosive atmospheres, such as normal atmospheric conditions.

The cyclic load resistance determined by the provisions of this Appendix is applicable only to structures subject to temperatures not exceeding 300 °F (150 °C).

The *engineer of record* shall provide either complete details including weld sizes or shall specify the planned cycle life and the maximum range of moments, shears and reactions for the *connections*.

3.2. CALCULATION OF MAXIMUM STRESSES AND STRESS RANGES

Calculated stresses shall be based upon *elastic analysis*. Stresses shall not be amplified by *stress concentration* factors for geometrical discontinuities.

For bolts and threaded rods subject to axial tension, the calculated stresses shall include the effects of *prying action*, if any. In the case of axial stress combined with bending, the maximum stresses, of each kind, shall be those determined for concurrent arrangements of the applied *load*.

For members having symmetric cross sections, the *fasteners* and welds shall be arranged symmetrically about the axis of the member, or the total stresses including those due to eccentricity shall be included in the calculation of the *stress* range.

For axially loaded angle members where the center of gravity of the connecting welds lies between the line of the center of gravity of the angle cross section and the center of the connected leg, the effects of eccentricity shall be ignored. If the center of gravity of the connecting welds lies outside this zone, the total stresses, including those due to *joint eccentricity*, shall be included in the calculation of *stress* range.

3.3. DESIGN STRESS RANGE

The range of *stress* at *service loads* shall not exceed the *design stress range* computed as follows.

(a) For stress categories A, B, B′, C, D, E and E′ the design stress range, F_{SR}, shall be determined by Equation A-3-1 or A-3-1M.

$$F_{SR} = \left(\frac{C_f}{N}\right)^{0.333} \geq F_{TH} \tag{A-3-1}$$

$$F_{SR} = \left(\frac{C_f \times 329}{N}\right)^{0.333} \geq F_{TH} \quad \text{(S.I.)} \tag{A-3-1M}$$

where

F_{SR} = design stress range, ksi (MPa)
C_f = constant from Table A-3.1 for the category
N = number of stress range fluctuations in design life
= number of stress range fluctuations per day × 365 × years of design life
F_{TH} = threshold *fatigue stress* range, maximum *stress* range for indefinite design life from Table A-3.1, ksi (MPa)

(b) For stress category F, the design stress range, F_{SR}, shall be determined by Equation A-3-2 or A-3-2M.

$$F_{SR} = \left(\frac{C_f}{N}\right)^{0.167} \geq F_{TH} \tag{A-3-2}$$

$$F_{SR} = \left(\frac{C_f \times 11 \times 10^4}{N}\right)^{0.167} \geq F_{TH} \quad \text{(S.I.)} \tag{A-3-2M}$$

(c) For tension-loaded plate elements connected at their end by cruciform, T, or corner details with *complete-joint-penetration* (CJP) *groove welds* or *partial-joint-penetration* (PJP) *groove welds*, *fillet welds*, or combinations of the preceding, transverse to the direction of stress, the design stress range on the cross section of the tension-loaded plate element at the toe of the weld shall be determined as follows:

(i) Based upon crack initiation from the toe of the weld on the tension loaded plate element the design stress range, F_{SR}, shall be determined by Equation A-3-3 or A-3-3M, for stress category C which is equal to

$$F_{SR} = \left(\frac{44 \times 10^8}{N}\right)^{0.333} \geq 10 \tag{A-3-3}$$

$$F_{SR} = \left(\frac{14.4 \times 10^{11}}{N}\right)^{0.333} \geq 68.9 \quad \text{(S.I.)} \tag{A-3-3M}$$

(ii) Based upon crack initiation from the root of the weld the design stress range, F_{SR}, on the tension loaded plate element using transverse PJP groove welds, with or without reinforcing or contouring fillet welds, the design stress range on the cross section at the toe of the weld shall be determined by Equation A-3-4 or A-3-4M, stress category C′ as follows:

$$F_{SR} = R_{PJP}\left(\frac{44 \times 10^8}{N}\right)^{0.333} \tag{A-3-4}$$

$$F_{SR} = R_{PJP}\left(\frac{14.4 \times 10^{11}}{N}\right)^{0.333} \quad \text{(S.I.)} \tag{A-3-4M}$$

where

R_{PJP} is the reduction factor for reinforced or nonreinforced transverse PJP groove welds determined as follows:

$$R_{PJP} = \left(\frac{0.65 - 0.59\left(\frac{2a}{t_p}\right) + 0.72\left(\frac{w}{t_p}\right)}{t_p^{0.167}}\right) \leq 1.0$$

$$R_{PJP} = \left(\frac{1.12 - 1.01\left(\frac{2a}{t_p}\right) + 1.24\left(\frac{w}{t_p}\right)}{t_p^{0.167}}\right) \leq 1.0 \qquad \text{(S.I.)}$$

If $R_{PJP} = 1.0$, use stress category C.

$2a$ = the length of the nonwelded root face in the direction of the thickness of the tension-loaded plate, in. (mm)

w = the leg size of the reinforcing or contouring fillet, if any, in the direction of the thickness of the tension-loaded plate, in. (mm)

t_p = thickness of tension loaded plate, in. (mm)

(iii) Based upon crack initiation from the roots of a pair of transverse fillet welds on opposite sides of the tension loaded plate element the design stress range, F_{SR}, on the cross section at the toe of the welds shall be determined by Equation A-3-5 or A-3-5M, stress category C″ as follows:

$$F_{SR} = R_{FIL}\left(\frac{44 \times 10^8}{N}\right)^{0.333} \quad \text{(A-3-5)}$$

$$F_{SR} = R_{FIL}\left(\frac{14.4 \times 10^{11}}{N}\right)^{0.333} \quad \text{(S.I.)} \quad \text{(A-3-5M)}$$

where

R_{FIL} is the reduction factor for joints using a pair of transverse fillet welds only.

$$R_{FIL} = \left(\frac{0.06 + 0.72\left(w/t_p\right)}{t_p^{0.167}}\right) \leq 1.0$$

$$R_{FIL} = \left(\frac{0.10 + 1.24\left(w/t_p\right)}{t_p^{0.167}}\right) \leq 1.0 \quad \text{(S.I.)}$$

If $R_{FIL} = 1.0$, use stress category C.

3.4. BOLTS AND THREADED PARTS

The range of *stress* at *service loads* shall not exceed the stress range computed as follows.

(a) For mechanically fastened *connections* loaded in shear, the maximum range of stress in the connected material at service loads shall not exceed the *design stress range* computed using Equation A-3-1 where C_f and F_{TH} are taken from Section 2 of Table A-3.1.

(b) For high-strength bolts, common bolts, and threaded anchor rods with cut, ground or rolled threads, the maximum range of tensile stress on the net tensile area from applied axial *load* and moment plus load due to *prying action* shall not exceed the design stress range computed using Equation A-3-1 or A-3-1M. The factor C_f shall be taken as 3.9×10^8 (as for stress category E′). The threshold stress, F_{TH} shall be taken as 7 ksi (48 MPa) (as for stress category D). The net tensile area is given by Equation A-3-6 and A-3-6M.

$$A_t = \frac{\pi}{4}\left(d_b - \frac{0.9743}{n}\right)^2 \quad \text{(A-3-6)}$$

$$A_t = \frac{\pi}{4}\left(d_b - 0.9382P\right)^2 \quad \text{(S.I.)} \quad \text{(A-3-6M)}$$

where

P = *pitch*, in. per thread (mm per thread)
d_b = the nominal diameter (body or shank diameter), in. (mm)
n = threads per in. (threads per mm)

For *joints* in which the material within the *grip* is not limited to steel or joints which are not tensioned to the requirements of Table J3.1 or J3.1M, all axial load and moment applied to the joint plus effects of any *prying action* shall be assumed to be carried exclusively by the bolts or rods.

For joints in which the material within the grip is limited to steel and which are tensioned to the requirements of Table J3.1 or J3.1M, an analysis of the relative *stiffness* of the connected parts and bolts shall be permitted to be used to determine the tensile *stress* range in the pretensioned bolts due to the total service live load and moment plus effects of any prying action. Alternatively, the stress range in the bolts shall be assumed to be equal to the stress on the net tensile area due to 20 percent of the absolute value of the service load axial load and moment from dead, live and other loads.

3.5. SPECIAL FABRICATION AND ERECTION REQUIREMENTS

Longitudinal backing bars are permitted to remain in place, and if used, shall be continuous. If splicing is necessary for long *joints*, the bar shall be joined with complete penetration butt joints and the reinforcement ground prior to assembly in the joint.

In transverse joints subject to tension, backing bars, if used, shall be removed and the joint back gouged and welded.

In transverse complete-joint-penetration T and corner joints, a reinforcing *fillet weld*, not less than $^1/_4$ in. (6 mm) in size shall be added at re-entrant corners.

The surface roughness of flame cut edges subject to significant cyclic tensile *stress* ranges shall not exceed 1,000 μin. (25 μm), where ASME B46.1 is the reference standard.

Reentrant corners at cuts, *copes* and weld access holes shall form a radius of not less than $^3/_8$ in. (10 mm) by predrilling or subpunching and reaming a hole, or by thermal cutting to form the radius of the cut. If the radius portion is formed by thermal cutting, the cut surface shall be ground to a bright metal surface.

For transverse butt joints in regions of high tensile stress, run-off tabs shall be used to provide for cascading the weld termination outside the finished joint. End dams shall not be used. Run-off tabs shall be removed and the end of the weld finished flush with the edge of the member.

See Section J2.2b for requirements for *end returns* on certain fillet welds subject to cyclic *service loading*.

TABLE A-3.1
Fatigue Design Parameters

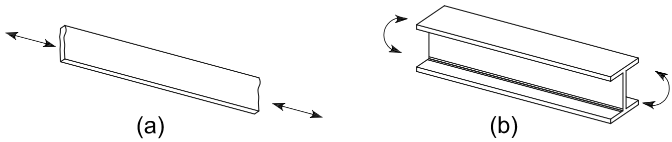
Description	Stress Category	Constant C_f	Threshold F_{TH} ksi (MPa)	Potential Crack Initiation Point
SECTION 1 – PLAIN MATERIAL AWAY FROM ANY WELDING				
1.1 Base metal, except non-coated weathering steel, with rolled or cleaned surface. Flame-cut edges with surface roughness value of 1,000 μin. (25 μm) or less, but without reentrant corners.	A	250×10^8	24 (165)	Away from all welds or structural *connections*
1.2 Non-coated weathering steel base metal with rolled or cleaned surface. Flame-cut edges with surface roughness value of 1,000 μin. (25 μm) or less, but without reentrant corners.	B	120×10^8	16 (110)	Away from all welds or structural *connections*
1.3 Member with drilled or reamed holes. Member with re-entrant corners at *copes*, cuts, block-outs or other geometrical discontinuities made to requirements of Appendix 3.5, except weld access holes.	B	120×10^8	16 (110)	At any external edge or at hole perimeter
1.4 Rolled cross sections with weld access holes made to requirements of Section J1.6 and Appendix 3.5. Members with drilled or reamed holes containing bolts for attachment of light bracing where there is a small longitudinal component of brace *force*.	C	44×10^8	10 (69)	At *reentrant* corner of weld access hole or at any small hole (may contain bolt for minor *connections*)
SECTION 2 – CONNECTED MATERIAL IN MECHANICALLY FASTENED JOINTS				
2.1 Gross area of base metal in lap joints connected by high-strength bolts in joints satisfying all requirements for slip-critical connections.	B	120×10^8	16 (110)	Through gross section near hole
2.2 Base metal at net section of high-strength bolted joints, designed on the basis of bearing resistance, but fabricated and installed to all requirements for slip-critical connections.	B	120×10^8	16 (110)	In net section originating at side of hole
2.3 Base metal at the net section of other mechanically fastened joints except eye bars and pin plates.	D	22×10^8	7 (48)	In net section originating at side of hole
2.4 Base metal at net section of *eyebar* head or pin plate.	E	11×10^8	4.5 (31)	In net section originating at side of hole

TABLE A-3.1 (cont.)
Fatigue Design Parameters

Illustrative Typical Examples

SECTION 1 – PLAIN MATERIAL AWAY FROM ANY WELDING

1.1 and 1.2



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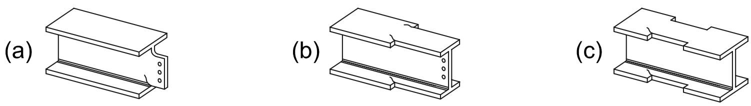
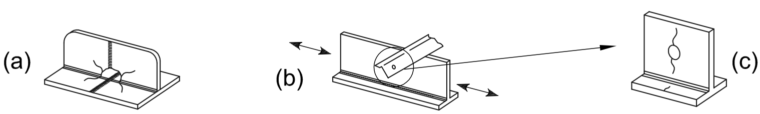
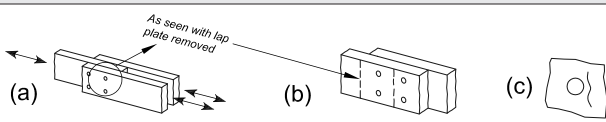


1.4



SECTION 2 – CONNECTED MATERIAL IN MECHANICALLY FASTENED JOINTS

2.1



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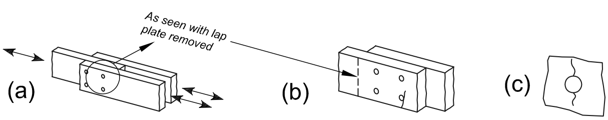

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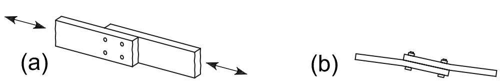

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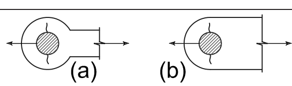


TABLE A-3.1 (cont.) Fatigue Design Parameters

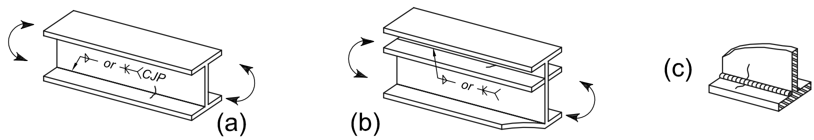
Description	Stress Category	Constant C_f	Threshold F_{TH} ksi (MPa)	Potential Crack Initiation Point
SECTION 3 – WELDED JOINTS JOINING COMPONENTS OF BUILT-UP MEMBERS				
3.1 Base metal and weld metal in members without attachments built-up of plates or shapes connected by continuous longitudinal complete-joint-penetration groove welds, back gouged and welded from second side, or by continuous fillet welds.	B	120×10^8	16 (110)	From surface or internal discontinuities in weld away from end of weld
3.2 Base metal and weld metal in members without attachments built-up of plates or shapes, connected by continuous longitudinal complete-joint-penetration groove welds with backing bars not removed, or by continuous partial-joint-penetration groove welds.	B′	61×10^8	12 (83)	From surface or internal discontinuities in weld, including weld attaching backing bars
3.3 Base metal and weld metal termination of longitudinal welds at weld access holes in connected built-up members.	D	22×10^8	7 (48)	From the weld termination into the web or flange
3.4 Base metal at ends of longitudinal intermittent fillet weld segments.	E	11×10^8	4.5 (31)	In connected material at start and stop locations of any weld deposit
3.5 Base metal at ends of partial length welded coverplates narrower than the flange having square or tapered ends, with or without welds across the ends of coverplates wider than the flange with welds across the ends.				In flange at toe of end weld or in flange at termination of longitudinal weld or in edge of flange with wide coverplates
Flange thickness $\leq$ 0.8 in. (20 mm)	E	11×10^8	4.5 (31)	
Flange thickness $>$ 0.8 in. (20 mm)	E′	3.9×10^8	2.6 (18)	
3.6 Base metal at ends of partial length welded coverplates wider than the flange without welds across the ends.	E′	3.9×10^8	2.6 (18)	In edge of flange at end of coverplate weld
SECTION 4 – LONGITUDINAL FILLET WELDED END CONNECTIONS				
4.1 Base metal at junction of axially loaded members with longitudinally welded end connections. Welds shall be on each side of the axis of the member to balance weld stresses.				Initiating from end of any weld termination extending into the base metal
$t \leq$ 0.8 in. (20 mm)	E	11×10^8	4.5 (31)	
$t >$ 0.8 in. (20 mm)	E′	3.9×10^8	2.6 (18)	

TABLE A-3.1 (cont.)
Fatigue Design Parameters

Illustrative Typical Examples

SECTION 3 – WELDED JOINTS JOINING COMPONENTS OF BUILT-UP MEMBERS

3.1



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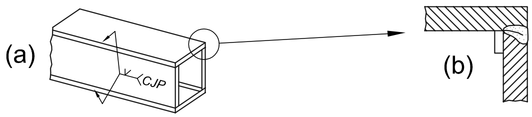
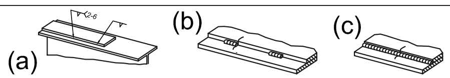


3.3

3.4



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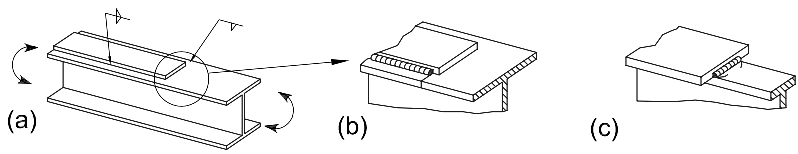
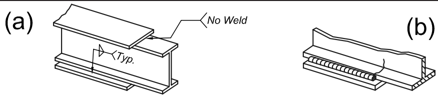


3.6



SECTION 4 – LONGITUDINAL FILLET WELDED END CONNECTIONS

4.1

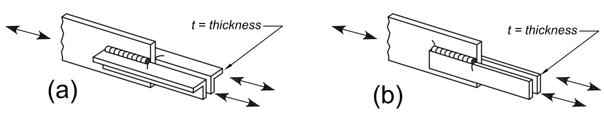


TABLE A-3.1 (cont.)
Fatigue Design Parameters

Description	Stress Category	Constant C_f	Threshold F_{TH} ksi (MPa)	Potential Crack Initiation Point
SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS				
5.1 Base metal and weld metal in or adjacent to complete-joint-penetration groove welded splices in rolled or welded cross sections with welds ground essentially parallel to the direction of stress.	B	120×10^8	16 (110)	From internal discontinuities in filler metal or along the fusion boundary
5.2 Base metal and weld metal in or adjacent to complete-joint-penetration groove welded splices with welds ground essentially parallel to the direction of stress at transitions in thickness or width made on a slope no greater than 8 to 20%.				From internal discontinuities in filler metal or along fusion boundary or at start of transition when $F_y \geq 90$ ksi (620 MPa)
$F_y < 90$ ksi (620 MPa)	B	120×10^8	16 (110)	
$F_y \geq 90$ ksi (620 MPa)	B′	61×10^8	12 (83)	
5.3 Base metal with F_y equal to or greater than 90 ksi (620 MPa) and weld metal in or adjacent to complete-joint-penetration groove welded splices with welds ground essentially parallel to the direction of stress at transitions in width made on a radius of not less than 2 ft (600 mm) with the point of tangency at the end of the groove weld.	B	120×10^8	16 (110)	From internal discontinuities in filler metal or discontinuities along the fusion boundary
5.4 Base metal and weld metal in or adjacent to the toe of complete-joint-penetration T or corner joints or splices, with or without transitions in thickness having slopes no greater than 8 to 20%, when weld reinforcement is not removed.	C	44×10^8	10 (69)	From surface discontinuity at toe of weld extending into base metal or along fusion boundary.
5.5 Base metal and weld metal at transverse end connections of tension-loaded plate elements using partial-joint-penetration butt or T or corner joints, with reinforcing or contouring fillets, F_{SR} shall be the smaller of the toe crack or root crack stress range.				Initiating from geometrical discontinuity at toe of weld extending into base metal or, initiating at weld root subject to tension extending up and then out through weld
Crack initiating from weld toe:	C	44×10^8	10 (69)	
Crack initiating from weld root:	C′	Eqn. A-3-4 or A-3-4M	None provided	

TABLE A-3.1 (cont.)
Fatigue Design Parameters

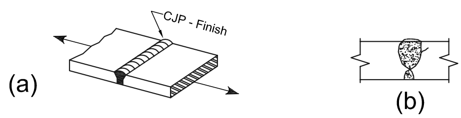
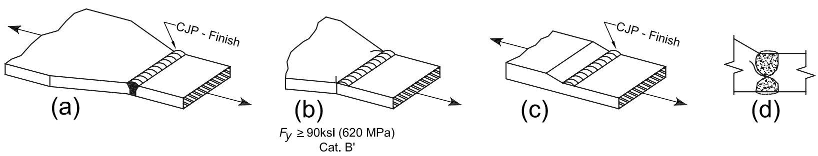
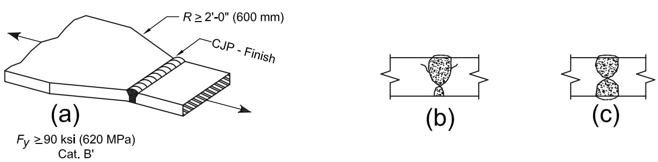
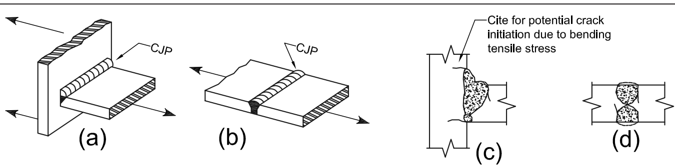
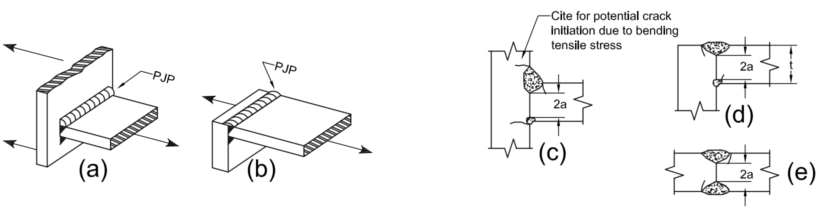
	Illustrative Typical Examples
SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS	
5.1	
5.2	
5.3	
5.4	
5.5	

TABLE A-3.1 (cont.)
Fatigue Design Parameters

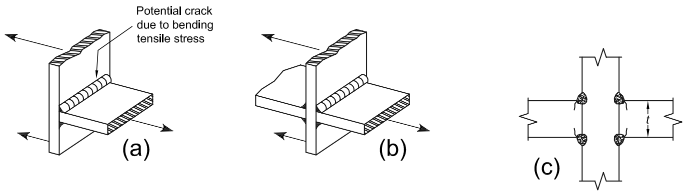
Description	Stress Category	Constant C_f	Threshold F_{TH} ksi (MPa)	Potential Crack Initiation Point
SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS (cont'd)				
5.6 Base metal and filler metal at transverse end connections of tension-loaded plate elements using a pair of fillet welds on opposite sides of the plate. F_{SR} shall be the smaller of the toe crack or root crack stress range.				Initiating from geometrical discontinuity at toe of weld extending into base metal or, initiating at *weld root* subject to tension extending up and then out through weld
Crack initiating from weld toe:	C	44×10^8	10 (69)	
Crack initiating from weld root:	C″	Eqn. A-3-5 or A-3-5M	None provided	
5.7 Base metal of tension loaded plate elements and on girders and rolled beam webs or flanges at toe of transverse fillet welds adjacent to welded transverse stiffeners.	C	44×10^8	10 (69)	From geometrical discontinuity at *toe of fillet* extending into base metal
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS				
6.1 Base metal at details attached by complete-joint-penetration groove welds subject to longitudinal loading only when the detail embodies a transition radius R with the weld termination ground smooth.				Near point of tangency of radius at edge of member
$R \geq$ 24 in. (600 mm)	B	120×10^8	16 (110)	
24 in. $>$ $R \geq$ 6 in. (600 mm $>$ $R \geq$ 150 mm)	C	44×10^8	10 (69)	
6 in. $>$ $R \geq$ 2 in. (150 mm $>$ $R \geq$ 50 mm)	D	22×10^8	7 (48)	
2 in. (50 mm) $>$ R	E	11×10^8	4.5 (31)	

TABLE A-3.1 (cont.)
Fatigue Design Parameters

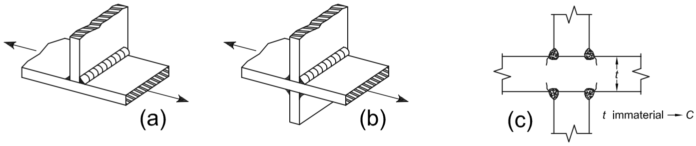
Illustrative Typical Examples

SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS (cont'd)

5.6



5.7



SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS

6.1

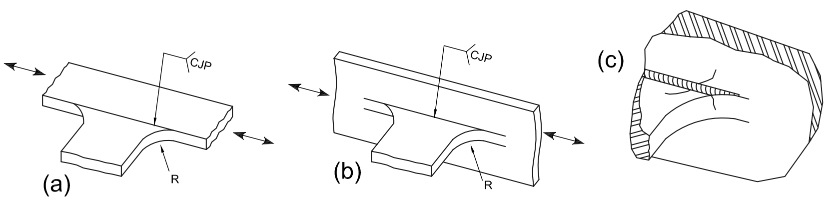


TABLE A-3.1 (cont.) Fatigue Design Parameters

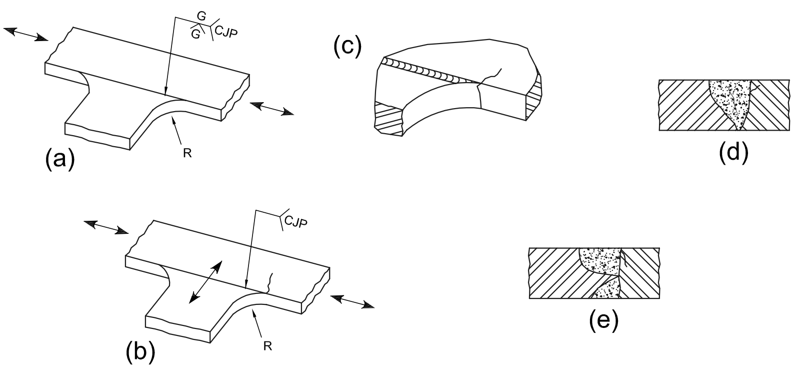
Description	Stress Category	Constant C_f	Threshold F_{TH} ksi (MPa)	Potential Crack Initiation Point
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)				
6.2 Base metal at details of equalthickness attached by complete-joint-penetration groove welds subject to transverse loading with or without longitudinal loading when the detail embodies a transition radius R with the weld termination ground smooth:				
When weld reinforcement is removed:				Near points of tangency of radius or in the weld or at fusion boundary or member or attachment
$R \geq$ 24 in. (600 mm)	B	120×10^8	16 (110)	
24 in. $>$ $R \geq$ 6 in. (600 mm $>$ $R \geq$ 150 mm)	C	44×10^8	10 (69)	
6 in. $>$ $R \geq$ 2 in. (150 mm $>$ $R \geq$ 50 mm)	D	22×10^8	7 (48)	
2 in. (50 mm) $>$ R	E	11×10^8	4.5 (31)	
When weld reinforcement is not removed:				At toe of the weld either along edge of member or the attachment
$R \geq$ 24 in. (600 mm)	C	44×10^8	10 (69)	
24 in. $>$ $R \geq$ 6 in. (600 mm $>$ $R \geq$ 150 mm)	C	44×10^8	10 (69)	
6 in. $>$ $R \geq$ 2 in. (150 mm $>$ $R \geq$ 50 mm)	D	22×10^8	7 (48)	
2 in. (50 mm) $>$ R	E	11×10^8	4.5 (31)	
6.3 Base metal at details of unequal thickness attached by complete-joint-penetration groove welds subject to transverse loading with or without longitudinal loading when the detail embodies a transition radius R with the weld termination ground smooth.				
When weld reinforcement is removed:				
$R >$ 2 in. (50 mm)	D	22×10^8	7 (48)	At toe of weld along edge of thinner material
$R \leq$ 2 in. (50 mm)	E	11×10^8	4.5 (31)	In weld termination in small radius
When reinforcement is not removed:				
Any radius	E	11×10^8	4.5 (31)	At toe of weld along edge of thinner material

TABLE A-3.1 (cont.)
Fatigue Design Parameters

Illustrative Typical Examples

SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)

6.2



6.3

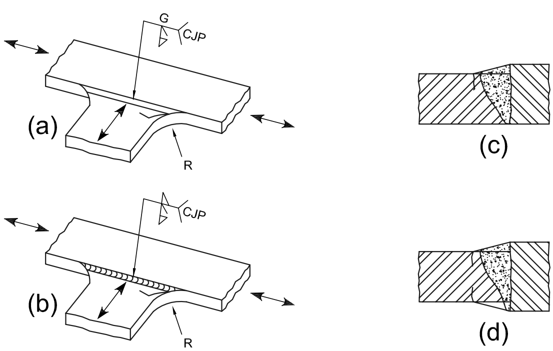


TABLE A-3.1 (cont.)
Fatigue Design Parameters

Description	Stress Category	Constant C_f	Threshold F_{TH} ksi (MPa)	Potential Crack Initiation Point
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)				
6.4 Base metal subject to longitudinal stress at transverse members, with or without transverse stress, attached by fillet or partial penetration groove welds parallel to direction of stress when the detail embodies a transition radius, *R*, with weld termination ground smooth:				In weld termination or from the toe of the weld extending into member
$R > 2$ in. (50 mm)	D	22×10^8	7 (48)	
$R \leq 2$ in. (50 mm)	E	11×10^8	4.5 (31)	
SECTION 7 – BASE METAL AT SHORT ATTACHMENTS[1]				
7.1 Base metal subject to longitudinal loading at details attached by fillet welds parallel or transverse to the direction of stress where the detail embodies no transition radius and with detail length in direction of stress, *a*, and attachment height normal to the surface of the member, *b*:				In the member at the end of the weld
$a < 2$ in. (50 mm)	C	44×10^8	10 (69)	
2 in. (50 mm) $\leq a \leq 12\,b$ or 4 in (100 mm)	D	22×10^8	7 (48)	
$a > 12b$ or 4 in. (100 mm) when b is ≤ 1 in. (25 mm)	E	11×10^8	4.5 (31)	
$a > 12b$ or 4 in. (100 mm) when b is > 1 in. (25 mm)	E′	3.9×10^8	2.6 (18)	
7.2 Base metal subject to longitudinal *stress* at details attached by fillet or partial-joint-penetration groove welds, with or without transverse *load* on detail, when the detail embodies a transition radius, *R*, with weld termination ground smooth:				In weld termination extending into member
$R > 2$ in. (50 mm)	D	22×10^8	7 (48)	
$R \leq 2$ in. (50 mm)	E	11×10^8	4.5 (31)	

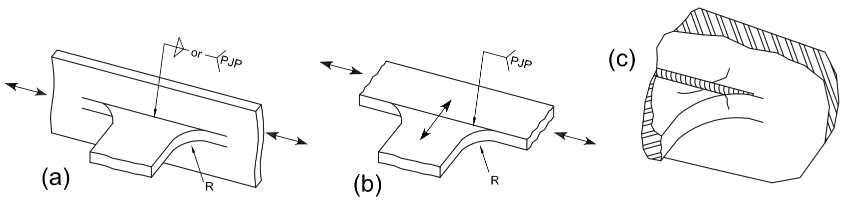
[1] "Attachment" as used herein, is defined as any steel detail welded to a member which, by its mere presence and independent of its loading, causes a discontinuity in the *stress* flow in the member and thus reduces the *fatigue* resistance.

TABLE A-3.1 (cont.)
Fatigue Design Parameters

Illustrative Typical Examples

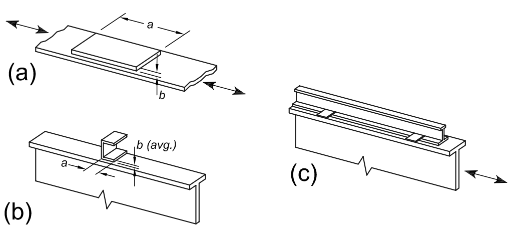
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)

6.4



SECTION 7 – BASE METAL AT SHORT ATTACHMENTS

7.1



7.2

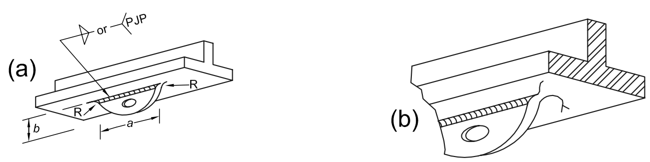


TABLE A-3.1 (cont.) Fatigue Design Parameters

Description	Stress Category	Constant C_f	Threshold F_{TH} ksi (MPa)	Potential Crack Initiation Point
SECTION 8 - MISCELLANEOUS				
8.1 Base metal at stud-type shear connectors attached by fillet or electric stud welding.	C	44×10^8	10 (69)	At toe of weld in base metal
8.2 Shear on throat of continuous or intermittent longitudinal or transverse fillet welds.	F	150×10^{10} (Eqn. A-3-2 or A-3-2M)	8 (55)	In throat of weld
8.3 Base metal at plug or slot welds.	E	11×10^8	4.5 (31)	At end of weld in base metal
8.4 Shear on plug or slot welds.	F	150×10^{10} (Eqn. A-3-2 or A-3-2M)	8 (55)	At *faying surface*
8.5 Not fully tightened high-strength bolts, common bolts, threaded anchor rods and hanger rods with cut, ground or rolled threads. Stress range on tensile stress area due to live load plus prying action when applicable.	E′	3.9×10^8	7 (48)	At the root of the threads extending into the tensile *stress* area

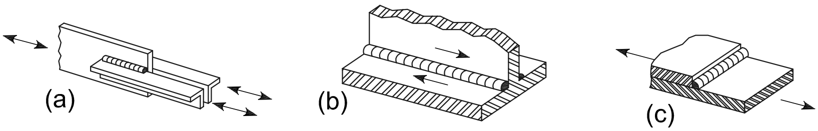
TABLE A-3.1 (cont.)
Fatigue Design Parameters

Illustrative Typical Examples

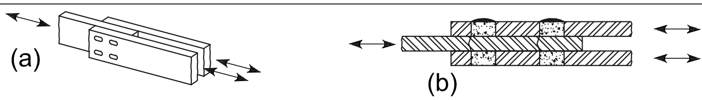
SECTION 8 – MISCELLANEOUS

8.1

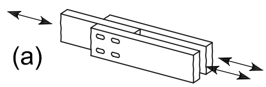
8.2



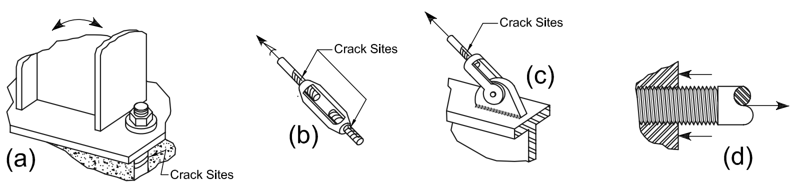
8.3



8.4



8.5



APPENDIX 4

STRUCTURAL DESIGN FOR FIRE CONDITIONS

This appendix provides criteria for the design and evaluation of *structural steel* components, systems and frames for fire conditions. These criteria provide for the determination of the heat input, thermal expansion and degradation in mechanical properties of materials at elevated temperatures that cause progressive decrease in strength and *stiffness* of structural components and systems at elevated temperatures.

The appendix is organized as follows:

4.1. GENERAL PROVISIONS

The methods contained in this appendix provide regulatory evidence of compliance in accordance with the design applications outlined in this section.

The appendix uses the following terms in addition to the terms in the Glossary.

Active fire protection: Building materials and systems that are activated by a fire to mitigate adverse effects or to notify people to take some action mitigate adverse effects.

Compartmentation: The enclosure of a building space with elements that have a specific fire endurance.

Convective heat transfer: The transfer of thermal energy from a point of higher temperature to a point of lower temperature through the motion of an intervening medium.

Design-basis fire: A set of conditions that define the development of a fire and the spread of combustion products throughout a building or portion thereof.

Elevated temperatures: Heating conditions experienced by building elements or structures as a result of fire, which are in excess of the anticipated ambient conditions.

Fire: Destructive burning, as manifested by any or all of the following: light, flame, heat, or smoke.

Fire barrier: Element of construction formed of fire-resisting materials and tested in accordance with ASTM Standard E119, or other approved standard fire resistance test, to demonstrate compliance with the Building Code.

Fire endurance: A measure of the elapsed time during which a material or assembly continues to exhibit fire resistance.

Fire resistance: That property of assemblies that prevents or retards the passage of excessive heat, hot gases or flames under conditions of use and enables them to continue to perform a stipulated function.

Fire resistance rating: The period of time a building element, component or assembly maintains the ability to contain a fire, continues to perform a given structural function, or both, as determined by test or methods based on tests.

Flashover: The rapid transition to a state of total surface involvement in a fire of combustible materials within an enclosure.

Heat flux: Radiant energy per unit surface area.

Heat release rate: The rate at which thermal energy is generated by a burning material.

Passive fire protection: Building materials and systems whose ability to resist the effects of fire does not rely on any outside activating condition or mechanism.

Performance-based design: An engineering approach to structural design that is based on agreed-upon performance goals and objectives, engineering analysis and quantitative assessment of alternatives against those design goals and objectives using accepted engineering tools, methodologies and performance criteria.

Prescriptive design: A design method that documents compliance with general criteria established in a building code.

Restrained construction: Floor and roof assemblies and individual *beams* in buildings where the surrounding or supporting structure is capable of resisting substantial thermal expansion throughout the range of anticipated elevated temperatures.

Unrestrained construction: Floor and roof assemblies and individual *beams* in buildings that are assumed to be free to rotate and expand throughout the range of anticipated elevated temperatures.

4.1.1. Performance Objective

Structural components, members and building frame systems shall be designed so as to maintain their load-bearing function during the design-basis fire and to satisfy other performance requirements specified for the building occupancy.

Deformation criteria shall be applied where the means of providing structural fire resistance, or the design criteria for fire barriers, requires consideration of the deformation of the load-carrying structure.

Within the compartment of fire origin, *forces* and deformations from the design-basis fire shall not cause a breach of horizontal or vertical compartmentation.

4.1.2. Design by Engineering Analysis

The analysis methods in Section 4.2 are permitted to be used to document the anticipated performance of steel framing when subjected to design-basis fire scenarios. Methods in Section 4.2 provide evidence of compliance with performance objectives established in Section 4.1.1.

The analysis methods in Section 4.2 are permitted to be used to demonstrate an equivalency for an alternative material or method, as permitted by the building code.

4.1.3. Design by Qualification Testing

The qualification testing methods in Section 4.3 are permitted to be used to document the fire resistance of steel framing subject to the standardized fire testing protocols required by building codes.

4.1.4. Load Combinations and Required Strength

The *required strength* of the structure and its elements shall be determined from the following *gravity load combination*:

$$[0.9 \text{ or } 1.2]D + T + 0.5L + 0.2S \qquad \text{(A-4-1)}$$

where

D = *nominal dead load*
L = nominal occupancy live load
S = nominal snow load
T = nominal *forces* and deformations due to the design-basis fire defined in Section 4.2.1

A lateral *notional load*, $N_i = 0.002Y_i$, as defined in Appendix 7.2, where N_i = notional *lateral load* applied at framing level i and Y_i = *gravity load* from combination A-4-1 acting on framing level i, shall be applied in combination with the loads stipulated in Equation A-4-1. Unless otherwise stipulated by the *authority having jurisdiction*, D, L and S shall be the *nominal loads* specified in ASCE 7.

4.2. STRUCTURAL DESIGN FOR FIRE CONDITIONS BY ANALYSIS

It is permitted to design structural members, components and building frames for elevated temperatures in accordance with the requirements of this section.

4.2.1. Design-Basis Fire

A design-basis fire shall be identified to describe the heating conditions for the structure. These heating conditions shall relate to the fuel commodities and compartment characteristics present in the assumed fire area. The fuel *load* density based on the occupancy of the space shall be considered when determining the total fuel *load*. Heating conditions shall be specified either in terms of a heat flux or temperature of the upper gas layer created by the fire. The variation of the heating conditions with time shall be determined for the duration of the fire.

When the analysis methods in Section 4.2 are used to demonstrate an equivalency as an alternative material or method as permitted by a building code, the design-basis fire shall be determined in accordance with ASTM E119.

4.2.1.1. Localized Fire

Where the heat release rate from the fire is insufficient to cause flashover, a localized fire exposure shall be assumed. In such cases, the fuel composition, arrangement of the fuel array and floor area occupied by the fuel shall be used to determine the radiant heat flux from the flame and smoke plume to the structure.

4.2.1.2. Post-Flashover Compartment Fires

Where the heat release rate from the fire is sufficient to cause flashover, a post-flashover compartment fire shall be assumed. The determination of the temperature versus time profile resulting from the fire shall include fuel *load*, ventilation characteristics to the space (natural and mechanical), compartment dimensions and thermal characteristics of the compartment boundary.

4.2.1.3. Exterior Fires

The exposure of exterior structure to flames projecting from windows or other wall openings as a result of a post-flashover compartment fire shall be considered along with the radiation from the interior fire through the opening. The shape and length of the flame projection shall be used along with the distance between the flame and the exterior steelwork to determine the heat flux to the steel. The method identified in Section 4.2.1.2 shall be used for describing the characteristics of the interior compartment fire.

4.2.1.4. Fire Duration

The fire duration in a particular area shall be determined by considering the total combustible mass, in other words, fuel *load* available in the space. In the case of either a localized fire or a post-flashover compartment fire, the time duration shall be determined as the total combustible mass divided by the mass loss rate, except where determined from Section 4.2.1.2.

4.2.1.5. Active Fire Protection Systems

The effects of active fire protection systems shall be considered when describing the design-basis fire.

Where automatic smoke and heat vents are installed in nonsprinklered spaces, the resulting smoke temperature shall be determined from calculation.

4.2.2. Temperatures in Structural Systems under Fire Conditions

Temperatures within structural members, components and frames due to the heating conditions posed by the design-basis fire shall be determined by a heat transfer analysis.

Table A-4.2.1
Properties of Steel at Elevated Temperatures

Steel Tempreature (°F)[°C]	$k_E = E_m/E$	$k_y = F_{ym}/F_y$	$k_u = F_{um}/F_y$
68 [20]	*	*	*
200 [93]	1.00	*	*
400 [204]	0.90	*	*
600 [316]	0.78	*	*
750 [399]	0.70	1.00	1.00
800 [427]	0.67	0.94	0.94
1000 [538]	0.49	0.66	0.66
1200 [649]	0.22	0.35	0.35
1400 [760]	0.11	0.16	0.16
1600 [871]	0.07	0.07	0.07
1800 [982]	0.05	0.04	0.04
2000 [1093]	0.02	0.02	0.02
2200 [1204]	0.00	0.00	0.00

*Use ambient properties.

4.2.3. Material Strengths at Elevated Temperatures

Material properties at elevated temperatures shall be determined from test data. In the absence of such data, it is permitted to use the material properties stipulated in this section. These relationships do not apply for steels with a *yield strength* in excess of 65 ksi (448 MPa) or concretes with specified compression strength in excess of 8,000 psi (55 MPa).

4.2.3.1. Thermal Elongation

Thermal expansion of structural and reinforcing steels: For calculations at temperatures above 150 °F (65 °C), the coefficient of thermal expansion shall be 7.8×10^{-6}/°F (1.4×10^{-5}/°C).

Thermal expansion of normal weight concrete: For calculations at temperatures above 150 °F (65 °C), the coefficient of thermal expansion shall be 1.0×10^{-5}/°F (1.8×10^{-5}/°C).

Thermal expansion of lightweight concrete: For calculations at temperatures above 150 °F (65 °C), the coefficient of thermal expansion shall be 4.4×10^{-6}/°F (7.9×10^{-6}/°C).

4.2.3.2. Mechanical Properties at Elevated Temperatures

The deterioration in strength and *stiffness* of structural members, components, and systems shall be taken into account in the *structural analysis* of the frame. The values F_{ym}, F_{um}, E_m, f'_{cm}, E_{cm} and ε_{cu} at elevated temperature to be used in *structural analysis*, expressed as the ratio with respect to the property at ambient, assumed to be 68 °F (20 °C), shall be defined as in Tables A-4.2.1 and A-4.2.2. It is permitted to interpolate between these values.

Table A-4.2.2
Properties of Concrete at Elevated Temperatures

Concrete Temperature (°F)[°C]	$k_c = f'_{cm}/f'_c$		E_{cm}/E_c	ε_{cu}(%)
	NWC	LWC		LWC
68 [20]	1.00	1.00	1.00	0.25
200 [93]	0.95	1.00	0.93	0.34
400 [204]	0.90	1.00	0.75	0.46
550 [288]	0.86	1.00	0.61	0.58
600 [316]	0.83	0.98	0.57	0.62
800 [427]	0.71	0.85	0.38	0.80
1000 [538]	0.54	0.71	0.20	1.06
1200 [649]	0.38	0.58	0.092	1.32
1400 [760]	0.21	0.45	0.073	1.43
1600 [871]	0.10	0.31	0.055	1.49
1800 [982]	0.05	0.18	0.036	1.50
2000 [1093]	0.01	0.05	0.018	1.50
2200 [1204]	0.00	0.00	0.00	–

For lightweight concrete (LWC), values of ε_{cu} shall be obtained from tests.

4.2.4. Structural Design Requirements

4.2.4.1. General Structural Integrity

The structural frame shall be capable of providing adequate strength and deformation capacity to withstand, as a system, the structural actions developed during the fire within the prescribed limits of deformation. The *structural system* shall be designed to sustain local damage with the structural system as a whole remaining stable.

Continuous *load* paths shall be provided to transfer all *forces* from the exposed region to the final point of resistance. The foundation shall be designed to resist the forces and to accommodate the deformations developed during the design-basis fire.

4.2.4.2. Strength Requirements and Deformation Limits

Conformance of the structural system to these requirements shall be demonstrated by constructing a mathematical model of the structure based on principles of structural mechanics and evaluating this model for the internal forces and deformations in the members of the structure developed by the temperatures from the design-basis fire.

Individual members shall be provided with adequate strength to resist the shears, axial forces and moments determined in accordance with these provisions.

Connections shall develop the strength of the connected members or the forces indicated above. Where the means of providing fire resistance requires the consideration of deformation criteria, the deformation of the structural system, or members thereof, under the design-basis fire shall not exceed the prescribed limits.

4.2.4.3. Methods of Analysis

4.2.4.3a. Advanced Methods of Analysis

The methods of analysis in this section are permitted for the design of all steel building structures for fire conditions. The design-basis fire exposure shall be that determined in Section 4.2.1. The analysis shall include both a thermal response and the mechanical response to the design-basis fire.

The *thermal response* shall produce a temperature field in each structural element as a result of the design-basis fire and shall incorporate temperature-dependent thermal properties of the structural elements and fire-resistive materials as per Section 4.2.2.

The*mechanical response* results in forces and deflections in the structural system subjected to the thermal response calculated from the design-basis fire. The mechanical response shall take into account explicitly the deterioration in strength and *stiffness* with increasing temperature, the effects of thermal expansions and large deformations. Boundary conditions and connection fixity must represent the proposed structural design. Material properties shall be defined as per Section 4.2.3.

The resulting analysis shall consider all relevant *limit states*, such as excessive deflections, connection fractures, and overall or *local buckling*.

4.2.4.3b. Simple Methods of Analysis

The methods of analysis in this section are applicable for the evaluation of the performance of individual members at elevated temperatures during exposure to fire.

The support and restraint conditions (forces, moments and boundary conditions) applicable at normal temperatures may be assumed to remain unchanged throughout the fire exposure.

(1) Tension members

It is permitted to model the thermal response of a tension element using a one-dimensional heat transfer equation with heat input as directed by the design-basis fire defined in Section 4.2.1.

The *design strength* of a tension member shall be determined using the provisions of Chapter D, with steel properties as stipulated in Section 4.2.3 and assuming a uniform temperature over the cross section using the temperature equal to the maximum steel temperature.

(2) Compression members

It is permitted to model the thermal response of a compression element using a one-dimensional heat transfer equation with heat input as directed by the design-basis fire defined in Section 4.2.1.

The design strength of a compression member shall be determined using the provisions of Chapter E with steel properties as stipulated in Section 4.2.3.

(3) Flexural members

It is permitted to model the thermal response of flexural elements using a one-dimensional heat transfer equation to calculate bottom flange temperature and to assume that this bottom flange temperature is constant over the depth of the member.

The design strength of a flexural member shall be determined using the provisions of Chapter F with steel properties as stipulated in Section 4.2.3.

(4) Composite floor members

It is permitted to model the thermal response of flexural elements supporting a concrete slab using a one-dimensional heat transfer equation to calculate bottom flange temperature. That temperature shall be taken as constant between the bottom flange and mid-depth of the web and shall decrease linearly by no more than 25 percent from the mid-depth of the web to the top flange of the *beam*.

The design strength of a *composite* flexural member shall be determined using the provisions of Chapter I, with reduced *yield stresses* in the steel consistent with the temperature variation described under thermal response.

4.2.4.4. Design Strength

The design strength shall be determined as in Section B3.3. The *nominal strength*, R_n, shall be calculated using material properties, as stipulated in Section 4.2.3, at the temperature developed by the design-basis fire.

4.3. DESIGN BY QUALIFICATION TESTING

4.3.1. Qualification Standards

Structural members and components in steel buildings shall be qualified for the rating period in conformance with ASTM E119. It shall be permitted to demonstrate compliance with these requirements using the procedures specified for steel construction in Section 5 of ASCE/SFPE 29.

4.3.2. Restrained Construction

For floor and roof assemblies and individual *beams* in buildings, a restrained condition exists when the surrounding or supporting structure is capable of resisting actions caused by thermal expansion throughout the range of anticipated elevated temperatures.

Steel beams, girders and frames supporting concrete slabs that are welded or bolted to integral framing members (in other words, *columns*, girders) shall be considered restrained construction.

4.3.3. Unrestrained Construction

Steel beams, girders and frames that do not support a concrete slab shall be considered unrestrained unless the members are bolted or welded to surrounding construction that has been specifically designed and detailed to resist actions caused by thermal expansion.

A steel member bearing on a wall in a single span or at the end span of multiple spans shall be considered unrestrained unless the wall has been designed and detailed to resist effects of thermal expansion.

APPENDIX 5

EVALUATION OF EXISTING STRUCTURES

This appendix applies to the evaluation of the strength and *stiffness* under static vertical (gravity) *loads* of existing structures by *structural analysis*, by *load* tests, or by a combination of *structural analysis* and *load* tests when specified by the engineer of record or in the contract documents. For such evaluation, the steel grades are not limited to those listed in Section A3.1. This appendix does not address *load* testing for the effects of seismic *loads* or moving *loads* (vibrations).

The Appendix is organized as follows:

5.1. General Provisions
5.2. Material Properties
5.3. Evaluation by Structural Analysis
5.4. Evaluation by Load Tests
5.5. Evaluation Report

5.1. GENERAL PROVISIONS

These provisions shall be applicable when the evaluation of an existing steel structure is specified for (a) verification of a specific set of design loadings or (b) determination of the *available strength* of a *load* resisting member or system. The evaluation shall be performed by *structural analysis* (Section 5.3), by *load* tests (Section 5.4), or by a combination of *structural analysis* and load tests, as specified in the contract documents. Where load tests are used, the *engineer of record* shall first analyze the structure, prepare a testing plan, and develop a written procedure to prevent excessive permanent deformation or catastrophic collapse during testing.

5.2. MATERIAL PROPERTIES

1. Determination of Required Tests

The *engineer of record* shall determine the specific tests that are required from Section 5.2.2 through 5.2.6 and specify the locations where they are required. Where available, the use of applicable project records shall be permitted to reduce or eliminate the need for testing.

2. Tensile Properties

Tensile properties of members shall be considered in evaluation by *structural analysis* (Section 5.3) or *load* tests (Section 5.4). Such properties shall include the *yield stress*, *tensile strength* and *percent elongation*. Where available, certified mill test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM A6/A6M or A568/A568M, as applicable, shall be permitted for this purpose. Otherwise, tensile tests shall be conducted in accordance with ASTM A370 from samples cut from components of the structure.

3. Chemical Composition

Where welding is anticipated for repair or modification of existing structures, the chemical composition of the steel shall be determined for use in preparing a welding procedure specification (WPS). Where available, results from certified mill test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM procedures shall be permitted for this purpose. Otherwise, analyses shall be conducted in accordance with ASTM A751 from the samples used to determine tensile properties, or from samples taken from the same locations.

4. Base Metal Notch Toughness

Where welded tension splices in heavy shapes and plates as defined in Section A3.1d are critical to the performance of the structure, the *Charpy V-Notch toughness s*hall be determined in accordance with the provisions of Section A3.1d. If the notch toughness so determined does not meet the provisions of Section A3.1d, the *engineer of record* shall determine if remedial actions are required.

5. Weld Metal

Where structural performance is dependent on existing welded *connections*, representative samples of *weld metal* shall be obtained. Chemical analysis and mechanical tests shall be made to characterize the weld metal. A determination shall be made of the magnitude and consequences of imperfections. If the requirements of AWS D1.1 are not met, the *engineer of record* shall determine if remedial actions are required.

6. Bolts and Rivets

Representative samples of bolts shall be inspected to determine markings and classifications. Where bolts cannot be properly identified visually, representative samples shall be removed and tested to determine *tensile strength* in accordance with ASTM F606 or ASTM F606M and the bolt classified accordingly. Alternatively, the assumption that the bolts are ASTM A307 shall be permitted. Rivets shall be assumed to be ASTM A502, Grade 1, unless a higher grade is established through documentation or testing.

5.3. EVALUATION BY STRUCTURAL ANALYSIS

1. Dimensional Data

All dimensions used in the evaluation, such as spans, *column* heights, member spacings, bracing locations, cross section dimensions, thicknesses and *connection* details, shall be determined from a field survey. Alternatively, when available, it shall be permitted to determine such dimensions from applicable project design or shop drawings with field verification of critical values.

2. Strength Evaluation

Forces (*load effects*) in members and connections shall be determined by *structural analysis* applicable to the type of structure evaluated. The load effects shall be determined for the *loads* and *factored load combinations* stipulated in Section B2.

The *available strength* of members and connections shall be determined from applicable provisions of Chapters B through K of this Specification.

3. Serviceability Evaluation

Where required, the deformations at *service loads* shall be calculated and reported.

5.4. EVALUATION BY LOAD TESTS

1. Determination of Load Rating by Testing

To determine the *load* rating of an existing floor or roof structure by testing, a test load shall be applied incrementally in accordance with the *engineer of record's* plan. The structure shall be visually inspected for signs of distress or imminent failure at each load level. Appropriate measures shall be taken if these or any other unusual conditions are encountered.

The tested strength of the structure shall be taken as the maximum applied test load plus the in-situ dead load. The live load rating of a floor structure shall be determined by setting the tested strength equal to $1.2D + 1.6L$, where D is the nominal dead load and L is the nominal live load rating for the structure. The nominal live load rating of the floor structure shall not exceed that which can be calculated using applicable provisions of the specification. For roof structures, L_r, S, or R as defined in the Symbols, shall be substituted for L. More severe *load combinations* shall be used where required by *applicable building codes*.

Periodic unloading shall be considered once the *service load* level is attained and after the onset of inelastic structural behavior is identified to document the amount of permanent set and the magnitude of the inelastic deformations. Deformations of the structure, such as member deflections, shall be monitored at critical locations during the test, referenced to the initial position before loading. It shall be demonstrated, while maintaining maximum test load for one hour that the deformation of the structure does not increase by more than 10 percent above that at the beginning of the holding period. It is permissible to repeat the sequence if necessary to demonstrate compliance.

Deformations of the structure shall also be recorded 24 hours after the test loading is removed to determine the amount of permanent set. Because the amount of acceptable permanent deformation depends on the specific structure, no limit is specified for permanent deformation at maximum loading. Where it is not feasible to load test the entire structure, a segment or zone of not less than one complete bay, representative of the most critical conditions, shall be selected.

2. Serviceability Evaluation

When load tests are prescribed, the structure shall be loaded incrementally to the service load level. Deformations shall be monitored for a period of one hour. The structure shall then be unloaded and the deformation recorded.

5.5. EVALUATION REPORT

After the evaluation of an existing structure has been completed, the *engineer of record* shall prepare a report documenting the evaluation. The report shall indicate whether the evaluation was performed by *structural analysis*, by *load* testing or by a combination of structural analysis and load testing. Furthermore, when testing is performed, the report shall include the loads and load combination used and the load-deformation and time-deformation relationships observed. All relevant information obtained from design drawings, mill test reports and auxiliary material testing shall also be reported. Finally, the report shall indicate whether the structure, including all members and *connections*, is adequate to withstand the *load effects*.

APPENDIX 6

STABILITY BRACING FOR COLUMNS AND BEAMS

This appendix addresses the minimum brace strength and *stiffness* necessary to provide member *strengths* based on the *unbraced length* between braces with an *effective length factor*, K, equal to 1.0.

The appendix is organized as follows:

6.1. General Provisions
6.2. Columns
6.3. Beams

User Note: The requirements for the stability of braced-frame systems are provided in Chapter C. The provisions in this appendix apply to bracing, intended to stabilize individual members.

6.1. GENERAL PROVISIONS

Bracing is assumed to be perpendicular to the members to be braced; for inclined or *diagonal bracing*, the brace strength (*force* or moment) and *stiffness* (force per unit displacement or moment per unit rotation) shall be adjusted for the angle of inclination. The evaluation of the stiffness furnished by a brace shall include its member and geometric properties, as well as the effects of *connections* and anchoring details.

Two general types of bracing systems are considered, relative and nodal. A *relative brace* controls the movement of the brace point with respect to adjacent braced points. A *nodal brace* controls the movement at the braced point without direct interaction with adjacent braced points. The *available strength* and stiffness of the bracing shall equal or exceed the required limits unless analysis indicates that smaller values are justified by analysis.

A *second-order analysis* that includes an initial out-of-straightness of the member to obtain brace strength and stiffness is permitted in lieu of the requirements of this appendix.

6.2. COLUMNS

It is permitted to brace an individual *column* at end and intermediate points along its length by either relative or nodal bracing systems. It is assumed that *nodal braces* are equally spaced along the column.

1. Relative Bracing

The required brace strength is

$$P_{br} = 0.004 P_r \tag{A-6-1}$$

The required brace *stiffness* is

$$\beta_{br} = \frac{1}{\phi}\left(\frac{2P_r}{L_b}\right) \text{(LRFD)} \qquad \beta_{br} = \Omega\left(\frac{2P_r}{L_b}\right) \text{(ASD)} \tag{A-6-2}$$

where

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

L_b = distance between braces, in. (mm)

For design according to Section B3.3 (LRFD)

P_r = *required axial compressive strength* using *LRFD load combinations*, kips (N)

For design according to Section B3.4 (ASD)

P_r = required axial compressive strength using *ASD load combinations*, kips (N)

2. Nodal Bracing

The required brace strength is

$$P_{br} = 0.01 P_r \tag{A-6-3}$$

The required brace stiffness is

$$\beta_{br} = \frac{1}{\phi}\left(\frac{8P_r}{L_b}\right) \text{(LRFD)} \qquad \beta_{br} = \Omega\left(\frac{8P_r}{L_b}\right) \text{(ASD)} \tag{A-6-4}$$

where

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

For design according to Section B3.3 (LRFD)

P_r = *required axial compressive strength* using *LRFD load combinations*, kips (N)

For design according to Section B3.4 (ASD)

P_r = required axial compressive strength using *ASD load combinations*, kips (N)

When L_b is less than L_q, where L_q is the maximum *unbraced length* for the required *column force* with K equal to 1.0, then L_b in Equation A-6-4 is permitted to be taken equal to L_q.

6.3. BEAMS

At points of support for *beams*, girders and trusses, restraint against rotation about their longitudinal axis shall be provided. Beam bracing shall prevent the relative displacement of the top and bottom flanges, in other words, twist of the section. Lateral *stability* of beams shall be provided by *lateral bracing*, *torsional bracing* or a combination of the two. In members subjected to *double curvature* bending, the inflection point shall not be considered a brace point.

1. Lateral Bracing

Bracing shall be attached near the compression flange, except for a cantilevered member, where an end brace shall be attached near the top (tension) flange. Lateral bracing shall be attached to both flanges at the brace point nearest the inflection point for beams subjected to double curvature bending along the length to be braced.

1a. Relative Bracing

The required brace strength is

$$P_{br} = 0.008 M_r C_d / h_o \tag{A-6-5}$$

The required brace *stiffness* is

$$\beta_{br} = \frac{1}{\phi}\left(\frac{4 M_r C_d}{L_b h_o}\right) \text{(LRFD)} \qquad \beta_{br} = \Omega\left(\frac{4 M_r C_d}{L_b h_o}\right) \text{(ASD)} \tag{A-6-6}$$

where

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

h_o = distance between flange centroids, in. (mm)
C_d = 1.0 for bending in *single curvature*; 2.0 for double curvature; $C_d = 2.0$ only applies to the brace closest to the inflection point
L_b = laterally *unbraced length*, in. (mm)

For design according to Section B3.3 (LRFD)

M_r = *required flexural strength* using *LRFD load combinations*, kip-in. (N-mm)

For design according to Section B3.4 (ASD)

M_r = required flexural strength using *ASD load combinations*, kip-in. (N-mm)

1b. Nodal Bracing

The required brace strength is

$$P_{br} = 0.02 M_r C_d / h_o \tag{A-6-7}$$

The required brace *stiffness* is

$$\beta_{br} = \frac{1}{\phi}\left(\frac{10 M_r C_d}{L_b h_o}\right) \text{(LRFD)} \qquad \beta_{br} = \Omega\left(\frac{10 M_r C_d}{L_b h_o}\right) \text{(ASD)} \tag{A-6-8}$$

where

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

For design according to Section B3.3 (LRFD)

M_r = required flexural strength using LRFD load combinations, kip-in. (N-mm)

For design according to Section B3.4 (ASD)

M_r = required flexural strength using ASD load combinations, kip-in. (N-mm)

When L_b is less than L_q, the maximum unbraced length for M_r, then L_b in Equation A-6-8 shall be permitted to be taken equal to L_q.

2. Torsional Bracing

It is permitted to provide either nodal or continuous *torsional bracing* along the *beam* length. It is permitted to attach the bracing at any cross-sectional location and it need not be attached near the compression flange. The *connection* between a torsional brace and the beam shall be able to support the required moment given below.

2a. Nodal Bracing

The required bracing moment is

$$M_{br} = \frac{0.024 M_r L}{n C_b L_b} \tag{A-6-9}$$

The required cross-frame or *diaphragm* bracing *stiffness* is

$$\beta_{Tb} = \frac{\beta_T}{\left(1 - \dfrac{\beta_T}{\beta_{sec}}\right)} \tag{A-6-10}$$

where

$$\beta_T = \frac{1}{\phi}\left(\frac{2.4 L M_r^2}{n E I_y C_b^2}\right) \text{ (LRFD)} \qquad \beta_T = \Omega\left(\frac{2.4 L M_r^2}{n E I_y C_b^2}\right) \text{ (ASD)} \tag{A-6-11}$$

$$\beta_{sec} = \frac{3.3E}{h_o}\left(\frac{1.5 h_o t_w^3}{12} + \frac{t_s b_s^3}{12}\right) \tag{A-6-12}$$

where

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 3.00 \text{ (ASD)}$$

> **User Note:** $\Omega = 1.5^2/\phi = 3.00$ in Equation A-6-11 because the moment term is squared.

L = span length, in. (mm)
n = number of *nodal braced* points within the span
E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)

I_y = out-of-plane moment of inertia, in.^{4} (mm^4)
C_b = modification factor defined in Chapter F
t_w = *beam* web thickness, in. (mm)
t_s = web *stiffener* thickness, in. (mm)
b_s = stiffener width for one-sided stiffeners (use twice the individual stiffener width for pairs of stiffeners), in. (mm)
β_T = brace *stiffness* excluding web distortion, kip-in./radian (N-mm/radian)
β_{sec} = web *distortional stiffness*, including the effect of web *transverse stiffeners*, if any, kip-in./radian (N-mm/radian)

For design according to Section B3.3 (LRFD)

M_r = *required flexural strength* using *LRFD load combinations*, kip-in. (N-mm)

For design according to Section B3.4 (ASD)

M_r = required flexural strength using *ASD load combinations*, kip-in. (N-mm)

If $\beta_{sec} < \beta_T$, Equation A-6-10 is negative, which indicates that torsional *beam* bracing will not be effective due to inadequate web distortional stiffness.

When required, the web stiffener shall extend the full depth of the braced member and shall be attached to the flange if the torsional brace is also attached to the flange. Alternatively, it shall be permissible to stop the stiffener short by a distance equal to $4t_w$ from any *beam* flange that is not directly attached to the torsional brace. When L_b is less than L_q, then L_b in Equation A-6-9 shall be permitted to be taken equal to L_q.

2b. Continuous Torsional Bracing

For continuous bracing, use Equations A-6-9, A-6-10 and A-6-13 with L/n taken as 1.0 and L_b taken as L_q; the bracing moment and stiffness are given per unit span length. The distortional stiffness for an unstiffened web is

$$\beta_{sec} = \frac{3.3Et_w^3}{12h_o} \qquad \text{(A-6-13)}$$

APPENDIX 7

DIRECT ANALYSIS METHOD

This appendix addresses the *direct analysis method* for structural systems comprised of *moment frames*, *braced frames*, *shear walls*, or combinations thereof.

The appendix is organized as follows:

7.1. GENERAL REQUIREMENTS

Members shall satisfy the provisions of Section H1 with the nominal *column* strengths, P_n, determined using $K = 1.0$. The *required strengths* for members, *connections* and other structural elements shall be determined using a second-order *elastic analysis* with the constraints presented in Section 7.3. All component and *connection* deformations that contribute to the lateral displacement of the structure shall be considered in the analysis.

7.2. NOTIONAL LOADS

Notional loads shall be applied to the lateral framing system to account for the effects of geometric imperfections, inelasticity, or both. *Notional loads* are *lateral loads* that are applied at each framing level and specified in terms of the *gravity loads* applied at that level. The *gravity load* used to determine the *notional load* shall be equal to or greater than the *gravity load* associated with the *load combination* being evaluated. *Notional loads* shall be applied in the direction that adds to the destabilizing effects under the specified *load combination*.

7.3. DESIGN-ANALYSIS CONSTRAINTS

(1) The *second-order analysis* shall consider both P-δ and P-Δ *effects*. It is permitted to perform the analysis using any general second-order analysis method, or by the amplified *first-order analysis* method of Section C2, provided that the B_1 and B_2 factors are based on the reduced *stiffness*es defined in Equations A-7-2 and A-7-3. Analyses shall be conducted according to the design and loading requirements specified in either Section B3.3 (LRFD) or Section B3.4 (ASD). For ASD, the second-order analysis shall be carried out under 1.6 times the ASD *load combinations* and the results shall be divided by 1.6 to obtain the *required strengths*.

Methods of analysis that neglect the effects of P-δ on the lateral displacement of the structure are permitted where the axial *loads* in all members whose flexural stiffnesses are considered to contribute to the lateral *stability* of the structure satisfy the following limit:

$$\alpha P_r < 0.15 P_{eL} \tag{A-7-1}$$

where

P_r = required axial compressive strength under LRFD or ASD *load combinations*, kips (N)
P_{eL} = $\pi^2 EI/L^2$, evaluated in the plane of bending

and

$$\alpha = 1.0 \text{ (LRFD)} \qquad \alpha = 1.6 \text{ (ASD)}$$

(2) A *notional load*, $N_i = 0.002Y_i$, applied independently in two orthogonal directions, shall be applied as a *lateral load* in all load combinations. This load shall be in addition to other lateral loads, if any,

where

N_i = notional lateral load applied at level i, kips (N)
Y_i = *gravity load* from the *LRFD load combination* or 1.6 times the ASD load combination applied at level i, kips (N)

The notional load coefficient of 0.002 is based on an assumed initial story out-of-plumbness ratio of 1/500. Where a smaller assumed out-of-plumbness is justified, the notional load coefficient may be adjusted proportionally.

For frames where the ratio of second-order drift to first-order drift is equal to or less than 1.5, it is permissible to apply the notional load, N_i, as a minimum lateral load for the gravity-only load combinations and not in combination with other lateral loads.

For all cases, it is permissible to use the assumed out-of-plumbness geometry in the analysis of the structure in lieu of applying a notional load or a minimum lateral load as defined above.

User Note: The unreduced stiffnesses (EI and AE) are used in the above calculations. The ratio of second-order drift to first-order drift can be represented by B_2, as calculated using Equation C2-3. Alternatively, the ratio can be calculated by comparing the results of a second-order analysis to the results of a first-order analysis, where the analyses are conducted either under LRFD load combinations directly or under ASD load combinations with a 1.6 factor applied to the ASD gravity loads.

(3) A reduced flexural *stiffness*, EI^*,

$$EI^* = 0.8\tau_b EI \tag{A-7-2}$$

shall be used for all members whose flexural stiffness is considered to contribute to the lateral *stability* of the structure,

where

I = moment of inertia about the axis of bending, in.^{4} (mm^4)

τ_b = 1.0 for $\alpha P_r / P_y \leq 0.5$

= $4[\alpha P_r / P_y\ (1 - \alpha P_r / P_y)]$ for $\alpha P_r / P_y > 0.5$

P_r = required axial compressive strength under *LRFD* or *ASD load combinations*, kips (N)

P_y = AF_y, member *yield strength*, kips (N)

and

$$\alpha = 1.0 \text{ (LRFD)} \qquad \alpha = 1.6 \text{ (ASD)}$$

In lieu of using $\tau_b < 1.0$ where $\alpha P_r / P_y > 0.5$, $\tau_b = 1.0$ may be used for all members, provided that an additive *notional load* of $0.001Y_i$ is added to the notional load required in (2).

(4) A reduced axial *stiffness*, EA^*,

$$EA^* = 0.8EA \tag{A-7-3}$$

shall be used for members whose axial stiffness is considered to contribute to the lateral *stability* of the structure, where A is the cross-sectional member area.

COMMENTARY

on the Specification for Structural Steel Buildings

March 9, 2005

(The Commentary is not a part of ANSI/AISC 360-05, *Specification for Structural Steel Buildings*, but is included for informational purposes only.)

INTRODUCTION

The Specification is intended to be complete for normal design usage.

The Commentary furnishes background information and references for the benefit of the design professional seeking further understanding of the basis, derivations and limits of the specification.

The Specification and Commentary are intended for use by design professionals with demonstrated engineering competence.

Commentary Glossary

The Commentary uses the following terms in addition to the terms defined in the Glossary of the Specification. Only the terms listed below are *italicized* where they first appear in the Commentary text.

Alignment chart. Nomograph for determining the effective length factor K for some types of columns.

Biaxial bending. Simultaneous bending of a member about two perpendicular axes.

Brittle fracture. Abrupt cleavage with little or no prior ductile deformation.

Column curve. Curve expressing the relationship between axial column strength and slenderness ratio.

Critical load. Load at which a perfectly straight member under compression may either assume a deflected position or may remain undeflected, or a beam under flexure may either deflect and twist out of plane or remain in its in-plane deflected position, as determined by a theoretical stability analysis.

Cyclic load. Repeatedly applied external load that may subject the structure to fatigue.

Drift damage index. Parameter used to measure the potential damage caused by interstory drift.

Effective moment of inertia. Moment of inertia of the cross section of a member that remains elastic when partial plastification of the cross section takes place, usually under the combination of residual stress and applied stress. Also, the moment of inertia based on effective widths of elements that buckle locally. Also, the moment of inertia used in the design of partially composite members.

Effective stiffness. Stiffness of a member computed using the effective moment of inertia of its cross section.

Fatigue threshold. Stress range at which fatigue cracking will not initiate regardless of the number of cycles of loading.

First order plastic analysis. *Structural analysis* based on the assumption of rigid-plastic behavior—in other words, that equilibrium is satisfied throughout the structure and the stress is at or below the yield stress—and in which equilibrium conditions are formulated on the undeformed structure.

Flexible connection. Connection permitting a portion, but not all, of the simple beam rotation of a member end.

Flexural-torsional buckling. Buckling mode in which a compression member bends and twists simultaneously without change in cross-sectional shape.

Inelastic action. Material deformation that does not disappear on removal of the force that produced it.

Inelastic strength. Strength of a structure or component after material has achieved the *yield stress* at sufficient locations that a strength *limit state* is reached.

Interstory drift. Lateral deflection of a floor relative to the lateral deflection of the floor immediately below, divided by the distance between floors, $(\delta_n - \delta_{n-1})/h)$.

Permanent load. *Load* in which variations over time are rare or of small magnitude. All other *loads* are *variable loads*.

Primary member. For ponding analysis, beam or girder that supports the concentrated reactions from the secondary members framing into it.

Residual stress. Stress that remains in an unloaded member after it has been formed into a finished product. (Examples of such stresses include, but are not limited to, those induced by cold bending, cooling after rolling or welding).

Rigid frame. Structure in which connections maintain the angular relationship between beam and column members under load.

Secondary member. For ponding analysis, beam or joist that directly supports the distributed ponding loads on the roof of the structure.

Sidesway. Lateral movement of a structure under the action of lateral loads, unsymmetrical vertical loads or unsymmetrical properties of the structure.

Sidesway buckling. Buckling mode of a multistory frame precipitated by the relative lateral displacements of joints, leading to failure by sidesway of the frame.

Squash load. Column area multiplied by the yield stress.

St. Venant torsion. Portion of the torsion in a member that induces only shear stresses in the member.

Strain hardening. Phenomenon wherein ductile steel, after undergoing considerable deformation at or just above yield point, exhibits the capacity to resist substantially higher loading than that which caused initial yielding.

Subassemblage. Truncated portion of a structural frame.

Tangent modulus. At any given stress level, the slope of the stress-strain curve of a material in the inelastic range as determined by the compression test of a small specimen under controlled conditions.

Total building drift. Lateral frame deflection at the top of the most occupied floor divided by the height of the building to that level, Δ/H.

Undercut. Notch resulting from the melting and removal of base metal at the edge of a weld.

Variable load. Load with substantial variation over time.

Warping torsion. Portion of the total resistance to torsion that is provided by resistance to warping of the cross section.

Yield plateau. Portion of the stress-strain curve for uniaxial tension or compression in which the stress remains essentially constant during a period of substantially increased strain.

CHAPTER A

GENERAL PROVISIONS

A1. SCOPE

The scope of this Specification is broader than that of the two AISC Specifications that it replaces: the 1999 *Load and Resistance Factor Design Specification for Structural Steel Buildings* (AISC, 2000b) and the 1989 *ASD Specification* (AISC, 1989). This Specification combines these two previous Specifications and incorporates the provisions of the *Load and Resistance Factor Design Specification for Steel Hollow Structural Sections* (AISC, 2000), the *Specification for Allowable Stress Design of Single-Angle Members* (AISC, 1989) and the *Load and Resistance Factor Design Specification for Single-Angle Members* (AISC, 2000a). The basic purpose of the provisions in this Specification is the determination of the available and nominal strength of the members, connections and other components of steel building structures. The nominal strength is usually defined in terms of resistance to a load effect, such as axial force, bending moment, shear or torque, but in some instances it is expressed in terms of a stress.

This Specification provides two methods of design:

(1) **Load and Resistance Factor Design (LRFD):** The nominal strength is multiplied by a resistance factor ϕ, and the resulting design strength is then required to equal or exceed the required strength determined by structural analysis for the appropriate LRFD load combination specified by the applicable building code.

(2) **Allowable Strength Design (ASD):** The nominal strength is divided by a safety factor Ω, and the resulting allowable strength is then required to equal or exceed the required strength determined by structural analysis for the appropriate ASD load combination specified by the applicable building code.

This Specification gives provisions for determining the values of the nominal strengths according to the applicable limit states and lists the corresponding values of the resistance factor ϕ and the safety factor Ω. The ASD safety factors are calibrated to give the same structural reliability and the same component size as the LRFD method at a live-to-dead load ratio of 3.

This Specification is applicable to both buildings and other structures. Many structures found in petrochemical plants, power plants, and other industrial applications are designed, fabricated and erected in a manner similar to buildings. It is not intended that this Specification address steel structures with vertical and lateral load-resisting systems that are not similar to buildings, such as those constructed of shells or catenary cables.

For the purposes of this Specification, HSS are defined as hollow structural sections with constant wall thickness and a round, square or rectangular cross section that is constant along the length of the member. HSS are manufactured by forming skelp (strip or plate) to the desired shape and joining the edges with a continuously welded seam. Published information is available describing the details of the various methods used to manufacture HSS (Graham, 1965; STI, 1996).

The *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2005) defines the practices that are the commonly accepted standards of custom and usage for structural steel fabrication and erection. As such, the *Code of Standard Practice* is primarily intended to serve as a contractual document to be incorporated into the contract between the buyer and seller of fabricated structural steel. Some parts of the *Code of Standard Practice*, however, form the basis for some of the provisions in this Specification. Therefore, the *Code of Standard Practice* is referenced in selected locations in this Specification to maintain the ties between these documents, where appropriate.

A2. REFERENCED SPECIFICATIONS, CODES AND STANDARDS

Section A2 provides references to documents cited in this Specification. Note that not all grades of a particular material specification are necessarily approved for use according to this Specification. For a list of approved materials and grades, see Section A3.

A3. MATERIAL

1. Structural Steel Materials

1a. ASTM Designations

There are hundreds of steel materials and products. This Specification lists those products/materials that are commonly useful to structural engineers and those that have a history of satisfactory performance. Other materials may be suitable for specific applications, but the evaluation of those materials is the responsibility of the engineer specifying them. In addition to typical strength properties, considerations for materials may include but are not limited to strength properties in transverse directions, ductility, formability, soundness, weldability including sensitivity to thermal cycles, notch toughness and other forms of crack sensitivity, coatings and corrosivity. Consideration for product form may include material considerations in addition to effects of production, tolerances, testing, reporting and surface profiles.

Hot-Rolled Structural Shapes. The grades of steel approved for use under this Specification, covered by ASTM specifications, extend to a yield stress of 100 ksi (690 MPa). Some of the ASTM specifications specify a minimum yield point, while others specify a minimum yield strength. The term "yield stress" is used in this Specification as a generic term to denote either the yield point or the yield strength.

It is important to be aware of limitations of availability that may exist for some combinations of strength and size. Not all structural section sizes are included in the various material specifications. For example, the 60 ksi (415 MPa) yield stress steel in the A572/A572M specification includes plate only up to $1^1/_4$ in. (32 mm) in thickness. Another limitation on availability is that even when a product is included in this Specification, it may be infrequently produced by the mills. Specifying these products may result in procurement delays or require ordering large quantities directly from the producing mills. Consequently, it is prudent to check availability before completing the details of a design. The AISC web site provides this information (www.aisc.org) and AISC's *Modern Steel Construction* publishes tables on availability twice per year.

Properties in the direction of rolling are of principal interest in the design of steel structures. Hence, yield stress as determined by the standard tensile test is the principal mechanical property recognized in the selection of the steels approved for use under this Specification. It must be recognized that other mechanical and physical properties of rolled steel, such as anisotropy, ductility, notch toughness, formability, corrosion resistance, etc., may also be important to the satisfactory performance of a structure.

It is not possible to incorporate in the Commentary adequate information to impart full understanding of all factors that might merit consideration in the selection and specification of materials for unique or especially demanding applications. In such a situation the user of the Specification is advised to make use of reference material contained in the literature on the specific properties of concern and to specify supplementary material production or quality requirements as provided for in ASTM material specifications. One such case is the design of highly restrained welded connections (AISC, 1973). Rolled steel is anisotropic, especially insofar as ductility is concerned; therefore, weld contraction strains in the region of highly restrained welded connections may exceed the strength of the material if special attention is not given to material selection, details, workmanship and inspection.

Another special situation is that of fracture control design for certain types of service conditions (AASHTO, 1998). For especially demanding service conditions such as structures exposed to low temperatures, particularly those with impact loading, the specification of steels with superior notch toughness may be warranted. However, for most buildings, the steel is relatively warm, strain rates are essentially static, and the stress intensity and number of cycles of full design stress are low. Accordingly, the probability of fracture in most building structures is low. Good workmanship and good design details incorporating joint geometry that avoids severe stress concentrations are generally the most effective means of providing fracture-resistant construction.

Hollow Structural Sections (HSS). Specified minimum tensile properties are summarized in Table C-A3.1 for various HSS and pipe material specifications and grades. ASTM A53 Grade B is included as an approved pipe material

TABLE C-A3.1
Minimum Tensile Properties of HSS and Pipe Steels

Specification	Grade	F_y, ksi (MPa)	F_u, ksi (MPa)
ASTM A53	**B**	35 (240)	60 (415)
ASTM A500 (round)	**A** **B** **C**	33 (228) 42 (290) 46 (317)	45 (311) 58 (400) 62 (428)
ASTM A500 (rectangular)	**A** **B** **C**	39 (269) 46 (317) 50 (345)	45 (311) 58 (400) 62 (428)
ASTM A501	**–**	36 (248)	58 (400)
ASTM A618 (round)	**I and II** **III**	50 (345) 50 (345)	70 (483) 65 (450)
ASTM A847	**–**	50 (345)	70 (483)
CAN/CSA-G40.20/G40.21	**350W**	51 (350)	65 (450)

specification because it is the most readily available round product in the United States. Other North American HSS products that have properties and characteristics that are similar to the approved ASTM products are produced in Canada under the *General Requirements for Rolled or Welded Structural Quality Steel* (CSA, 2003). In addition, pipe is produced to other specifications that meet the strength, ductility and weldability requirements of the materials in Section A3, but may have additional requirements for notch toughness or pressure testing.

Pipe can be readily obtained in ASTM A53 material and round HSS in ASTM A500 Grade B is also common. For rectangular HSS, ASTM A500 Grade B is the most commonly available material and a special order would be required for any other material. Depending upon size, either welded or seamless round HSS can be obtained. In North America, however, all ASTM A500 rectangular HSS for structural purposes are welded. Rectangular HSS differ from box sections in that they have uniform thickness except for some thickening in the rounded corners.

ASTM A500 Grade A material does not meet the ductility "limit of applicability" for direct connections in Section K2.3a(12). This limit requires that $F_y/F_u \leq 0.8$. In determining that other materials meet the ductility limit, it is important to note that ASTM A500 permits the yield strength to be determined by either the 0.2 percent offset method or at 0.5 percent elongation under load (EUL). Since ASTM A500 materials are cold-formed and have rounded stress-strain curves with no *yield plateau*, the latter method indicates yield strengths greater than the 0.2 percent offset. The ductility limit is intended to apply to yield strengths determined by the 0.2 percent offset. However, mill reports may indicate the EUL yield, raising concerns that the material does not have adequate ductility. Supplemental tension tests may be required to determine the 0.2 percent offset yield strength.

Even though ASTM A501 includes rectangular HSS, hot-formed rectangular HSS are not currently produced in the United States. The *General Requirements for Rolled or Welded Structural Quality Steel* (CSA, 2003) includes Class C (cold-formed) and Class H (cold-formed and stress relieved) HSS. Class H HSS have relatively low levels of *residual stress*, which enhances their performance in compression and may provide better ductility in the corners of rectangular HSS.

1c. Rolled Heavy Shapes

The web-to-flange intersection and the web center of heavy hot-rolled shapes, as well as the interior portions of heavy plates, may contain a more coarse grain structure and/or lower notch toughness material than other areas of these products. This is probably caused by ingot segregation, the somewhat lesser deformation during hot rolling, higher finishing temperature, and the slower cooling rate after rolling for these heavy sections. This characteristic is not detrimental to suitability for compression members or for nonwelded members. However, when heavy cross sections are joined by splices or connections using complete-joint-penetration welds that extend through the coarser and/or lower notch-tough interior portions, tensile strains induced by weld shrinkage may result in cracking. An example is a complete-joint-penetration groove welded connection of a heavy cross section beam to any column section. When members of lesser thickness are joined by complete-joint-penetration groove welds, which induce smaller weld shrinkage strains, to the finer grained and/or more notch-tough surface material of ASTM A6/A6M shapes and heavy built-up cross sections, the potential for cracking is significantly lower. An example is a complete-joint-penetration groove welded connection of a nonheavy cross-section beam to a heavy cross-section column.

For critical applications such as primary tension members, material should be specified to provide adequate notch toughness at service temperatures. Because of differences in the strain rate between the Charpy V-Notch (CVN) impact test and the strain rate experienced in actual structures, the CVN test is conducted at a temperature higher than the anticipated service temperature for the structure. The location of the CVN test specimens ("alternate core location") is specified in ASTM A6/A6M, Supplemental Requirement S30.

The notch toughness requirements of Section A3.1c are intended only to provide material of reasonable notch toughness for ordinary service applications. For unusual applications and/or low temperature service, more restrictive requirements and/or notch toughness requirements for other section sizes and thicknesses may be appropriate. To minimize the potential for fracture, the notch toughness requirements of Section A3.1c must be used in conjunction with good design and fabrication procedures. Specific requirements are given in Sections J1.5, J1.6, J2.6 and J2.7.

For rotary-straightened W-shapes, an area of reduced notch toughness has been documented in a limited region of the web immediately adjacent to the flange. This region may exist in W-shapes of all weights, not just heavy shapes. Considerations in design and detailing that recognize this situation are presented in Chapter J.

2. Steel Castings and Forgings

There are a number of ASTM specifications for steel castings. The SFSA *Steel Castings Handbook* (SFSA, 1995) recommends ASTM A216 as a product useful for steel structures. In addition to the requirements of this Specification, SFSA recommends that various other requirements be considered for cast steel products. It may be appropriate to inspect the first piece cast using magnetic particle inspection in accordance with ASTM E125, degree 1a, b, or c. Radiographic inspection level III may be desirable for critical sections of the first piece cast. Ultrasonic testing (UT) in compliance with ASTM E609 may be appropriate for first cast piece over 6 in. thick. Design approval, sample approval, periodic nondestructive testing of the mechanical properties, chemical testing, and selection of the correct welding specification should be among the issues defined in the selection and procurement of cast steel products. Refer to SFSA (1995) for design information about cast steel products.

3. Bolts, Washers and Nuts

The ASTM standard specification for A307 bolts covers two grades of fasteners. Either grade may be used under this Specification; however, it should be noted that Grade B is intended for pipe flange bolting and Grade A is the grade long in use for structural applications.

4. Anchor Rods and Threaded Rods

ASTM F1554 is the primary specification for anchor rods. Since there is a limit on the maximum available length of ASTM A325/A325M and ASTM A490/A490M bolts, the attempt to use these bolts for anchor rods with design lengths longer than the maximum available lengths has presented problems in the past. The inclusion of ASTM A449 and A354 materials in this Specification allows the use of higher strength material for bolts longer than ASTM A325/A325M and ASTM A490/A490M bolts.

The engineer of record should specify the required strength for threaded rods used as load-carrying members.

5. Filler Metal and Flux for Welding

The AWS Filler Metal Specifications listed in Section A3.5 are general specifications that include filler metal classifications suitable for building construction, as well as classifications that may not be suitable for building construction. The AWS D1.1, *Structural Welding Code Steel* (AWS, 2004) lists in Table 3.1 various electrodes that may be used for prequalified welding procedure specifications, for the various steels that are to be joined. This list specifically does not include various classifications of filler metals that are not suitable for structural steel applications. Filler metals listed under the various AWS A5 specifications may or may not have specified notch toughness properties, depending on the specific electrode classification. Section J2.6 identifies certain welded joints where notch toughness of filler metal is needed in building construction. There may be other situations where the engineer of record may elect to specify the use of filler metals with specified notch toughness properties, such as for structures subject to high loading rate, cyclic loading or seismic loading. Since AWS D1.1 does not automatically require that the filler metal used have specified notch toughness properties, it is important that filler metals used for such applications be of an AWS classification where such properties are required. This information can be found in the AWS Filler Metal Specifications and is often contained on the filler metal manufacturer's certificate of conformance or product specification sheets.

When specifying filler metal and/or flux by AWS designation, the applicable standard specifications should be carefully reviewed to assure a complete understanding of the designation reference. This is necessary because the AWS designation systems are not consistent. For example, in the case of electrodes for shielded metal arc welding (AWS A5.1), the first two or three digits indicate the nominal tensile strength classification, in ksi, of the filler metal and the final two digits indicate the type of coating. For metric designations, the first two digits times 10 indicate the nominal tensile strength classification in MPa. In the case of mild steel electrodes for submerged arc welding (AWS A5.17), the first one or two digits times 10 indicate the nominal tensile strength classification for both U.S. customary and metric units, while the final digit or digits times 10 indicate the testing temperature in degrees F, for filler metal impact tests. In the case of low-alloy steel covered arc welding electrodes (AWS A5.5), certain portions of the designation indicate a requirement for stress relief, while others indicate no stress relief requirement.

Engineers do not, in general, specify the exact filler metal to be employed on a particular structure. Rather, the decision as to which welding process and which filler metal are to be utilized is usually left with the fabricator or erector. Codes restrict the usage of certain filler materials, or impose qualification testing to prove the suitability of the specific electrode, so as to make certain that the proper filler metals are used.

A4. STRUCTURAL DESIGN DRAWINGS AND SPECIFICATIONS

The abbreviated list of requirements in this Specification is intended to be compatible with and a summary of the more extensive requirements in Section 3 of the *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2005). The user should refer to Section 3 of the *Code of Standard Practice for Steel Buildings and Bridges* for further information.

CHAPTER B

DESIGN REQUIREMENTS

B1. GENERAL PROVISIONS

Previous editions of the Specification contained a section entitled “Types of Construction,” for example, Section A2 in the 1999 *LRFD Specification* (AISC, 2000b). In this Specification there is no such section and the requirements related to “types of construction” have been divided between Section B1, Section B3.6, and Section J1.

Historically, “Types of Construction” was the section that established what type of structures the Specification covers. The preface to the 1999 *LRFD Specification* (AISC, 2000b) suggests that the purpose of the Specification is “to provide design criteria for routine use and not to provide specific criteria for infrequently encountered problems.” The preface to the 1978 *Specification* (AISC, 1978) contains similar language. While “routine use” may be difficult to describe, the contents of “Types of Construction” have been clearly directed at ordinary building frames with beams, columns and their connections.

The 1969 *Specification* (AISC, 1969) classified “types of construction” as Type 1, 2 or 3. The primary distinction among these three types of construction was the nature of the connections of the beams to the columns. Type 1 construction comprised “*rigid frames*,” now called moment-resisting frames that had connections capable of transmitting moment. Type 2 construction comprised “simple frames” with no moment transfer between beams and columns. Type 3 construction comprised “semi-rigid frames.” Type 3 construction used partially restrained connections and was allowed if a predictable and reliable amount of connection flexibility and moment transfer was demonstrable.

The 1986 *LRFD Specification* (AISC, 1986) changed the designation from Type 1, 2 or 3 to the designations FR (Fully Restrained) and PR (Partially Restrained). In these designations the term “restraint” refers to the degree of moment transfer and the associated deformation in the connections. The 1986 *LRFD Specification* also used the term “simple framing” to refer to structures with “simple connections,” that is, connections with negligible moment transfer. In essence, FR was equivalent to Type 1, “simple framing” was equivalent to Type 2, and PR was equivalent to Type 3 construction.

Type 2 construction of earlier specifications and “simple framing” of the 1986 *LRFD Specification* had additional provisions that allowed the wind loads to be carried by moment resistance of selected joints of the frame provided that:

(1) The connections and connected members have capacity to resist the wind moments;
(2) The girders are adequate to carry the full gravity load as "simple beams"; and
(3) The connections have adequate inelastic rotation capacity to avoid overstress of the fasteners or welds under combined gravity and wind loading.

The justification of considering the so-called "wind connections" as both simple (for gravity loads) and moment resisting (for wind loads) was provided in Sourochnikoff (1950) and Disque (1964). The basic argument asserts that the connections actually have some moment resistance but that the strength is low enough that under wind loads the connections would sustain inelastic deformations. Under repeated wind loads, then, the connection response would "shake down" to a condition wherein the moments in the connections under gravity loads would be very small but the elastic resistance of the connections to wind moments would remain the same as the initial resistance. These additional provisions for Type 2 construction have been used successfully for many years. More recent recommendations for this type of system are provided in Geschwindner and Disque (2005).

Section B1 widens the purview of this Specification to a broader class of construction types. It recognizes that a structural system is a combination of members connected in such a way that the structure can respond in different ways to meet different design objectives under different loads. Even within the purview of ordinary buildings, there can be an enormous variety in the design details.

This Specification is still meant to be primarily applicable to the common types of building frames with gravity loads carried by beams and girders and lateral loads carried by moment frames, braced frames or shear walls. However, there are many unusual buildings for which this Specification is also applicable. Rather than to attempt to establish the purview of the Specification with an exhaustive classification of construction types, Section B1 requires that the design of members and their connections be consistent with the intended use of the structure and the assumptions made in the analysis of the structure.

B2. LOADS AND LOAD COMBINATIONS

The loads and load combinations for this Specification are given in the applicable building code. In the absence of a specific local, regional or national building code, the load combinations and the nominal loads (for example, D, L, L_r, S, R, W and E) are the loads specified in Sections 3 through 9 of SEI/ASCE 7, *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2002). The latest 2002 edition of SEI/ASCE 7 has adopted, in most aspects, the seismic design provisions from the NEHRP *Recommended Provisions* (NEHRP, 1997), as have the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2002). The reader is referred to the commentaries of these documents for an expanded discussion on loads, load factors and seismic design.

This Specification permits design for strength by either LRFD or ASD.

LRFD Load Combinations. If LRFD is selected, the load combination requirements are defined in Section 2.3 of SEI/ASCE 7, while if ASD is selected, the load combination requirements are defined in Section 2.4 of that standard. In either case, it is assumed that the nominal loads—D, L, L_r, S, R, W and E—are as specified in Sections 3 through 9 of SEI/ASCE 7, or their equivalent, as stipulated by the authority having jurisdiction. The engineer should understand that the bases for the load combinations in Sections 2.3 and 2.4 of SEI/ASCE 7 are different.

The load combinations in Section 2.3 of SEI/ASCE 7 are based on modern probabilistic load modeling and a comprehensive survey of reliabilities inherent in traditional design practice (Galambos, Ellingwood, MacGregor, and Cornell, 1982; Ellingwood, MacGregor, Galambos, and Cornell, 1982). These load combinations utilize a "principal action-companion action format," which is based on the notion that the maximum combined load effect occurs when one of the time-varying loads takes on its maximum lifetime value (principal action) while the other *variable loads* are at "arbitrary point-in-time" values (companion actions), the latter being loads that would be measured in a load survey at any arbitrary time. The dead load, which is considered to be permanent, is the same for all combinations in which the load effects are additive. Research has shown that this approach to load combination analysis is consistent with the manner in which loads actually combine on structural elements and systems in situations in which strength limit states may be approached. The load factors reflect uncertainty in individual load magnitudes and in the analysis that transforms load to load effect. The nominal loads in Sections 3 through 9 of SEI/ASCE 7 are substantially in excess of the arbitrary point-in-time values. The nominal live, wind and snow loads historically have been associated with mean return periods of approximately 50 years, while the nominal earthquake effect in NEHRP (1997) is associated with a mean return period of approximately 2,500 years. To avoid having to specify both a maximum and an arbitrary point-in-time value for each load type, some of the specified load factors are less than unity in SEI/ASCE 7 combinations (2) through (5).

Load combinations (6) and (7) of SEI/ASCE 7, Section 2.3, apply specifically to cases in which the structural actions due to lateral forces and gravity loads counteract one another. In that case, where the dead load stabilizes the structure, the load factor on dead load is 0.9.

ASD Load Combinations. The load combinations in Section 2.4 of SEI/ASCE 7 for ASD are similar to those that have been used in allowable stress design for the past four decades. In ASD, safety is provided by the safety factor, Ω, and the nominal loads in the basic combinations (1) through (3) are not factored. The reduction in the combined time-varying load effect in combinations (4) and (6) is achieved by the load combination factor 0.75. This load combination factor dates back to the 1972 edition of ANSI Standard A58.1, the predecessor of SEI/ASCE 7. It should be noted that in SEI/ASCE 7, the 0.75 factor applies *only* to combinations of *variable loads*; it is irrational to reduce the dead load because it is *always* present and does not fluctuate in time. The load factor $0.6D$ in load combinations (7) and (8) in Section 2.4 of SEI/ASCE 7 addresses the situation in which the effects of lateral or uplift forces counteract the effect of gravity loads. This eliminates a deficiency in the traditional treatment of counteracting loads in allowable stress design and emphasizes the importance of checking stability. The earthquake load effect is multiplied by 0.7 in combinations (5) and (8) to align allowable strength design for earthquake effects with the definition of E in Section 9 of SEI/ASCE 7 which is based on strength principles.

The load combinations in Sections 2.3 and 2.4 of SEI/ASCE 7 apply only to design for strength limit states. Neither of these account for gross error or negligence.

Serviceability Load Combinations. Serviceability limit states and associated load factors are covered in Appendix B of SEI/ASCE 7. That Appendix contains a number of suggested load combinations for checking serviceability. While the nominal loads appearing in those equations are defined in Sections 3 through 7 of SEI/ASCE 7, the performance objectives for serviceability checking are different from those for checking strength, and thus the combinations and load factors are different.

B3. DESIGN BASIS

Load and Resistance Factor Design (LRFD) and Allowable Strength Design (ASD) are distinct methods. They are equally acceptable by this Specification, but their provisions are not identical and not interchangeable. Indiscriminate use of combinations of the two methods could result in design error. For these reasons they are specified as alternatives. There are, however, circumstances in which the two methods could be used in the design, modification or renovation of a structural system without conflicting, such as providing modifications to a structural floor system of an older building after assessing the as-built conditions.

1. Required Strength

This Specification permits the use of elastic, inelastic or plastic structural analysis. Generally, design is performed by elastic analysis. Provisions for inelastic and plastic analysis are given in Appendix 1. The required strength is determined by the appropriate methods of structural analysis.

In some circumstances, as in the proportioning of stability bracing members that carry no calculated forces (see, for example, Appendix 6), the required strength is explicitly stated in this Specification.

2. Limit States

A limit state is a condition in which a structural system or component becomes unfit for its intended purpose, when it is exceeded. Limit states may be dictated by functional requirements, such as maximum deflections or drift; they may be related to structural behavior, such as the formation of a plastic hinge or mechanism;

or they may represent the collapse of the whole or part of the structure, such as by instability or fracture. The design provisions provided make certain that the probability of reaching a limit state is acceptably small by stipulating the combination of load factors, resistance or safety factors, nominal loads and nominal strengths consistent with the design assumptions.

Two kinds of limit states apply to structures: (1) strength limit states define safety against local or overall failure conditions during the intended life of the structure; and (2) serviceability limit states define functional requirements. This Specification, like other structural design codes, primarily focuses on strength limit states because of overriding considerations of public safety. This does not mean that limit states of serviceability are not important to the designer, who must provide for functional performance and economy of design. However, serviceability considerations permit more exercise of judgment on the part of the designer.

Strength limit states vary from element to element, and several limit states may apply to a given element. The following strength limit states are the most common: yielding, formation of a plastic hinge, member or overall frame instability, lateral-torsional buckling, local buckling, rupture and fatigue. The most common serviceability limit states include unacceptable elastic deflections and drift, unacceptable vibrations, and permanent deformations.

3. Design for Strength Using Load and Resistance Factor Design (LRFD)

Design for strength by LRFD is performed in accordance with Equation B3-1. The left side of Equation B3-1, R_u, represents the required strength computed by structural analysis based on loads stipulated in SEI/ASCE 7 (ASCE, 2002), Section 2.3 (or their equivalent), while the right side, ϕR_n, represents the limiting structural resistance, or *design strength*, provided by the member.

The resistance factor ϕ in this Specification is equal to or less than 1.0. When compared to the nominal strength, R_n, computed according to the methods given in Chapters D through K, a ϕ-value of less than 1.0 accounts for inaccuracies of the theory and variations in mechanical properties and dimensions of members and frames. For limit states where $\phi = 1.0$, the nominal strength is judged to be sufficiently conservative when compared to the actual strength that no further reduction is needed.

The LRFD provisions are based on: (1) probabilistic models of loads and resistance; (2) a calibration of the LRFD provisions to the 1978 edition of the ASD Specification for selected members; and (3) the evaluation of the resulting provisions by judgment and past experience aided by comparative design office studies of representative structures.

In the probabilistic basis for LRFD (Ravindra and Galambos, 1978; Ellingwood and others, 1982), the load effects Q and the resistances R are modeled as statistically independent random variables. In Figure C-B3.1, relative frequency distributions for Q and R are portrayed as separate curves on a common plot for a hypothetical case. As long as the resistance R is greater than (to the right of) the effects of the loads Q, a margin of safety for the particular limit state exists. However, because Q and R are random variables, there is a small probability that R may be less than Q, in other words, $R < Q$. The probability of this limit state is related to the degree of overlap of the frequency distributions in Figure C-B3.1, which depends on their relative positioning (R_m *versus* Q_m) and their dispersions.

The probability that R is less than Q depends on the distribution shapes of each of the many variables (material, loads, etc.) that determine resistance and total load effect. Often, only the means and the standard deviations or coefficients of variation of the many variables involved in the makeup of R and Q can be estimated. However, this information is sufficient to build an approximate design provision that is independent of the knowledge of these distributions, by stipulating the following design condition:

$$\beta\sqrt{V_R^2 + V_Q^2} \leq \ln\left(R_m/Q_m\right) \tag{C-B3-1}$$

In this equation, R_m and Q_m are the mean values and V_R and V_Q are the coefficients of variation, respectively, of the resistance R and the load effect Q. For structural elements and the usual loading, R_m, Q_m, and the coefficients of variation, V_R and V_Q, can be estimated, so a calculation of

$$\beta = \frac{\ln\left(R_m/Q_m\right)}{\sqrt{V_R^2 + V_Q^2}} \tag{C-B3-2}$$

will give a comparative value of the measure of reliability of a structure or component. The parameter, β, is denoted the "safety" or "reliability" index.

Extensions to the determination of β in Equation C-B3-2 to accommodate additional probabilistic information and more complex design situations are described in Ellingwood and others (1982) and have been used in the development of the recommended load combinations in SEI/ASCE 7.

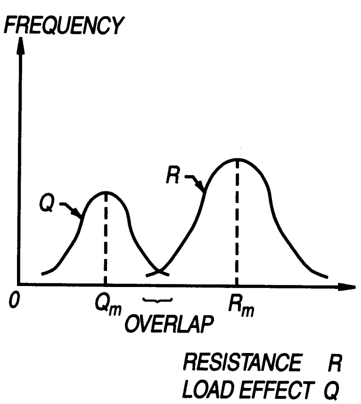


Fig. C-B3.1. Frequency distribution of load effect Q and resistance R.

The original studies for the statistical properties (mean values and coefficients of variation) used to develop the LRFD provisions for the basic material properties and for steel beams, columns, composite beams, plate girders, beam-columns and connection elements are presented in a series of eight articles in the September 1978 issue of the *Journal of the Structural Division*, ASCE (Vol. 104, ST9). The corresponding load statistics are given in Galambos and others (1982). Based on these statistics, the values of β inherent in the 1978 *Specification* (AISC, 1978) were evaluated under different load combinations (live/dead, wind/dead, etc.) and for various tributary areas for typical members (beams, columns, beam-columns, structural components, etc.). As might be expected, there was a considerable variation in the range of β-values. For example, compact rolled beams (flexure) and tension members (yielding) had β-values that decreased from about 3.1 at $L/D = 0.50$ to 2.4 at $L/D = 4$. This decrease is a result of ASD applying the same factor to dead load, which is relatively predictable, and live load, which is more variable. For bolted or welded connections, β was on the order of 4 to 5.

The variation of β that was inherent to ASD is reduced substantially in LRFD by specifying several target β-values and selecting load and resistance factors to meet these targets. The Committee on Specifications set the point at which LRFD is calibrated to ASD at $L/D = 3.0$ for braced compact beams in flexure and tension members at yield. The resistance factor, ϕ, for these limit states is 0.90, and the implied β is approximately 2.6 for members and 4.0 for connections. The larger β-value for connections reflects the fact that connections are expected to be stronger than the members they connect. Limit states for other members are handled similarly.

The databases on steel strength used in previous editions of the *LRFD Specification* were based mainly on research conducted prior to 1970. An important recent study of the material properties of structural shapes (Bartlett, Dexter, Graeser, Jelinek, Schmidt, and Galambos, 2003) addressed changes in steel production methods and steel materials that have occurred over the past 15 years. It was concluded that the new steel material characteristics did not warrant changes in the ϕ-values.

4. Design for Strength Using Allowable Strength Design (ASD)

The ASD method is provided in this Specification as an equal alternative to LRFD for use by engineers who prefer to deal with ASD load combinations and allowable stresses in the traditional ASD format. The term “allowable strength” has been introduced to emphasize that the basic equations of structural mechanics that underlie the provisions are the same for LRFD and ASD. This represents a departure from the past when LRFD and ASD were governed by separate specifications.

Traditional ASD is based on the concept that the maximum stress in a component shall not exceed a certain allowable stress under normal service conditions. The load effects are determined on the basis of an elastic analysis of the structure, while the allowable stress is the limiting stress (at yielding, instability, fracture, etc.) divided by a safety factor. The magnitude of the safety factor and the resulting allowable stress depend on the particular governing limit state against which the design must produce a certain margin of safety. For any single element, there may be a number of different allowable stresses that must be checked.

The safety factor in traditional ASD provisions was a function of both the material and the component being considered. It may have been influenced by factors such as member length, member behavior, load source and anticipated quality of workmanship. The traditional safety factors were based solely on experience and have remained unchanged for over 50 years. Although ASD-designed structures have performed adequately over the years, the actual level of safety provided was never known. This was the prime drawback of the traditional ASD approach. An illustration of typical performance data is provided in Bjorhovde (1978), where theoretical and actual safety factors for columns are examined.

Design for strength by ASD is performed in accordance with Equation B3-2. The ASD method provided in the Specification recognizes that the controlling modes of failure are the same for structures designed by ASD and LRFD. Thus, the nominal strength that forms the foundation of LRFD is the same nominal strength that provides the foundation for ASD. When considering available strength, the only difference between the two methods is the resistance factor in LRFD, ϕ, and the safety factor in ASD, Ω.

In developing appropriate values of Ω for use in this Specification, the aim was to assure similar levels of safety and reliability for the two methods. A straight forward approach for relating the resistance factor and the safety factor was developed. As already mentioned, the original LRFD Specification was calibrated to the 1978 *ASD Specification* at a live load to dead load ratio of 3. Thus, by equating the designs for the two methods at a ratio of live-to-dead load of 3, the relationship between ϕ and Ω can be determined. Using the live plus dead load combinations, with $L = 3D$, yields

$$\text{For LRFD}: \quad \phi R_n = 1.2D + 1.6L = 1.2D + 1.6 \times 3D = 6D \tag{C-B3-3}$$

$$R_n = \frac{6D}{\phi}$$

$$\text{For ASD}: \quad \frac{R_n}{\Omega} = D + L = D + 3D = 4D \tag{C-B3-4}$$

$$R_n = \frac{4D}{\Omega}$$

Equating R_n from the LRFD and ASD formulations and solving for Ω yields

$$\Omega = \frac{6D}{\phi} \times \frac{1}{4D} = \frac{1.5}{\phi} \tag{C-B3-5}$$

A similar approach was used to obtain the majority of values of Ω throughout the Specification.

5. Design for Stability

Section B3.5 provides the charging language for Chapter C on design for stability.

6. Design of Connections

Section B3.6 provides the charging language for Chapter J on the design of connections. Chapter J covers the proportioning of the individual elements of a connection (angles, welds, bolts, etc.) once the load effects on the connection are known. Section B3.6 establishes that the modeling assumptions associated with the structural analysis must be consistent with the conditions used in Chapter J to proportion the connecting elements.

In many situations, it is not necessary to include the connection elements as part of the analysis of the structural system. For example, simple and FR connections may often be idealized as pinned or fixed, respectively, for the purposes of structural analysis. Once the analysis has been completed, the deformations or forces computed at the joints may be used to proportion the connection elements. The classification of FR (fully restrained) and simple connections is meant to justify these idealizations for analysis with the provision that if, for example, one assumes a connection to be FR for the purposes of analysis, then the actual connection must meet the FR conditions. In other words, it must have adequate strength and stiffness, as described in the provisions and discussed below.

In certain cases, the deformation of the connection elements affects the way the structure resists load and hence the connections must be included in the analysis of the structural system. These connections are referred to as partially restrained (PR) moment connections. For structures with PR connections, the connection flexibility must be estimated and included in the structural analysis, as described in the following sections. Once the analysis is complete, the load effects and deformations computed for the connection can be used to check the adequacy of the connecting elements.

For simple and FR connections, the connection proportions are established after the final analysis of the structural design is completed, thereby greatly simplifying the design cycle. In contrast, the design of PR connections (like member selection) is inherently iterative because one must assume values of the connection proportions in order to establish the force-deformation characteristics of the connection needed to perform the structural analysis. The life-cycle performance characteristics (shakedown) must also be considered. The adequacy of the assumed proportions of the connection elements can be verified once the outcome of the structural analysis is known. If the connection elements are inadequate, then the values must be revised and the structural analysis repeated. The potential benefits of using PR connections for various types of framing systems are discussed extensively in the literature [for example, Lorenz, Kato, and Chen (1993); Leon (1994)].

Connection Classification. The basic assumption made in classifying connections is that the most important behavioral characteristics of the connection can be modeled by a moment-rotation (M-θ) curve. Figure C-B3.2 shows a typical M-θ

curve. Implicit in the moment-rotation curve is the definition of the connection as being a region of the column and beam along with the connecting elements. The connection response is defined this way because the rotation of the member in a physical test is generally measured over a gage length that incorporates the contributions of not only the connecting elements, but also the ends of the members being connected and the column panel zone.

Examples of connection classification schemes include those in Bjorhovde, Colson, and Brozzetti (1990) and Eurocode 3 (1992). These classifications account directly for the stiffness, strength and ductility of the connections.

Connection Stiffness. Because the nonlinear behavior of the connection manifests itself even at low moment-rotation levels, the initial stiffness of the connection K_i (shown in Figure C-B3.2) does not adequately characterize connection response at service levels. Furthermore, many connection types do not exhibit a reliable initial stiffness, or it exists only for a very small moment-rotation range. The secant stiffness K_S at service loads is taken as an index property of connection stiffness. Specifically, $K_S = M_S/\theta_S$ where M_S and θ_S are the moment and rotation, respectively, at service loads. In the discussion below, L and EI are the length and bending rigidity, respectively, of the beam.

If $K_S L/EI \geq 20$, then it is acceptable to consider the connection to be fully restrained (in other words, able to maintain the angles between members). If $K_S L/EI \leq 2$, then it is acceptable to consider the connection to be simple (in other words, rotates without developing moment). Connections with stiffnesses between these two limits are partially restrained and the stiffness, strength and ductility of the connection must be considered in the design (Leon, 1994). Examples of FR, PR and simple connection response curves are shown in Figure C-B3.3. The solid dot θ_S reflects the service load level and thereby defines the secant stiffness.

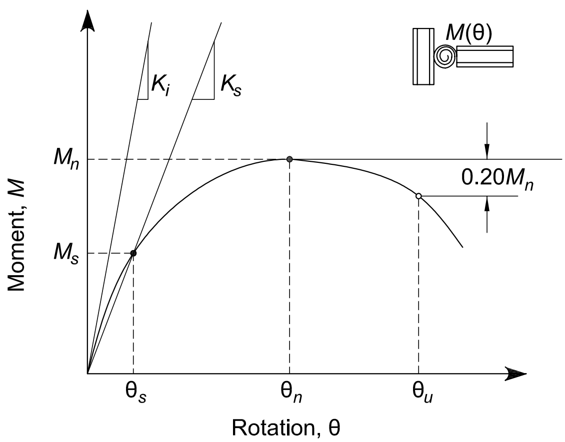


Fig. C-B3.2. Definition of stiffness, strength and ductility characteristics of the moment-rotation response of a partially restrained connection.

Connection Strength. The strength of a connection is the maximum moment that it is capable of carrying M_n, as shown in Figure C-B3.2. The strength of a connection can be determined on the basis of an ultimate limit-state model of the connection, or from a physical test. If the moment-rotation response does not exhibit a peak load then the strength can be taken as the moment at a rotation of 0.02 radian (Hsieh and Deierlein, 1991; Leon, Hoffman, and Staeger, 1996).

It is also useful to define a lower limit on strength below which the connection may be treated as a simple connection. Connections that transmit less than 20 percent of the fully plastic moment of the beam at a rotation of 0.02 radian may be considered to have no flexural strength for design. However, it should be recognized that the aggregate strength of many weak connections can be important when compared to that of a few strong connections (FEMA, 1997).

In Figure C-B3.3, the grey dot M_n indicates the maximum strength and the associated rotation θ_n. The open dot θ_u is the maximum rotation capacity. Note that it is possible for an FR connection to have a strength less than the strength of the beam. It is also possible for a PR connection to have a strength greater than the strength of the beam.

The strength of the connection must be adequate to resist the moment demands implied by the design loads.

Connection Ductility. If the connection strength substantially exceeds the fully plastic moment of the beam, then the ductility of the structural system is controlled by the beam and the connection can be considered elastic. If the connection strength only marginally exceeds the fully plastic moment of the beam, then the connection may experience substantial inelastic deformation before the beam reaches its full strength. If the beam strength exceeds the connection strength,

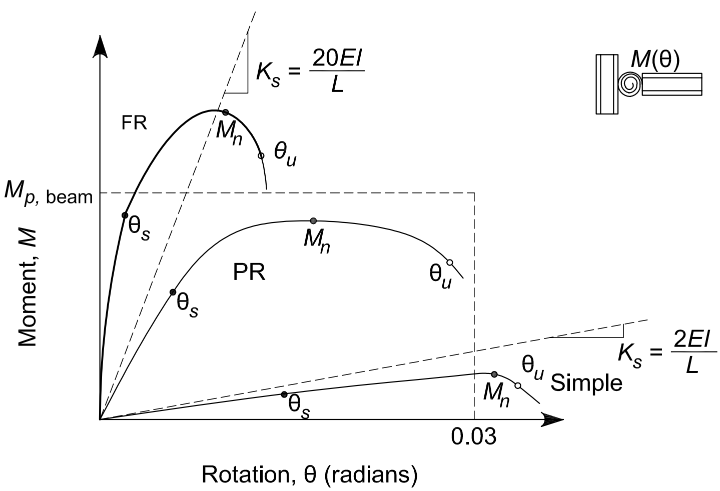


Fig. C-B3.3. Classification of moment-rotation response of fully restrained (FR), partially restrained (PR) and simple connections.

then deformations can concentrate in the connection. The ductility required of a connection will depend upon the particular application. For example, the ductility requirement for a braced frame in a nonseismic area will generally be less than the ductility required in a high seismic area. The rotation ductility requirements for seismic design depend upon the structural system (AISC, 2002).

In Figure C-B3.2, the rotation capacity, q_u, can be defined as the value of the connection rotation at the point where either (a) the resisting strength of the connection has dropped to $0.8M_n$ or (b) the connection has deformed beyond 0.03 radian. This second criterion is intended to apply to connections where there is no loss in strength until very large rotations occur. It is not prudent to rely on these large rotations in design.

The available rotation capacity, θ_u, should be compared with the rotation required at the strength limit state, as determined by an analysis that takes into account the nonlinear behavior of the connection. (Note that for design by ASD, the rotation required at the strength limit state should be assessed using analyses conducted at 1.6 times the ASD load combinations.) In the absence of an accurate analysis, a rotation capacity of 0.03 radian is considered adequate. This rotation is equal to the minimum beam-to-column connection capacity as specified in the seismic provisions for special moment frames (AISC, 2002). Many types of PR connections, such as top and seat-angle details, meet this criterion.

Structural Analysis and Design. When a connection is classified as PR the relevant response characteristics of the connection must be included in the analysis of the structure to determine the member and connection forces, displacements and the frame stability. Therefore, PR construction requires, first, that the moment-rotation characteristics of the connection be known and, second, that these characteristics be incorporated in the analysis and member design.

Typical moment-rotation curves for many PR connections are available from one of several databases [for example, Goverdhan (1983); Ang and Morris (1984); Nethercot (1985); and Kishi and Chen (1986)]. Care should be exercised when utilizing tabulated moment-rotation curves not to extrapolate to sizes or conditions beyond those used to develop the database since other failure modes may control (ASCE Task Committee on Effective Length, 1997). When the connections to be modeled do not fall within the range of the databases, it may be possible to determine the response characteristics from tests, simple component modeling, or finite element studies (FEMA, 1995). Examples of procedures to model connection behavior are given in the literature (Bjorhovde, Brozzetti, and Colson, 1988; Chen and Lui, 1991; Bjorhovde, Colson, Haaijer, and Stark, 1992; Lorenz and others, 1993; Chen and Toma, 1994; Chen, Goto, and Liew, 1995; Bjorhovde, Colson, and Zandonini, 1996; Leon, Hoffman, and Staeger, 1996; Leon and Easterling, 2002).

The degree of sophistication of the analysis depends on the problem at hand. Usually, design for PR construction requires separate analyses for the serviceability and strength limit states. For serviceability, an analysis using linear springs with a stiffness given by K_S (see Figure C-B3.2) is sufficient if the resistance demanded of the connection is well below the strength. When subjected to strength load combinations, a more careful procedure is needed so that the characteristics assumed in the analysis are consistent with those of the connection response. The response is especially nonlinear as the applied moment approaches the connection strength. In particular, the effect of the connection nonlinearity on second-order moments and other stability checks need to be considered (ASCE Task Committee on Effective Length, 1997).

7. Design for Serviceability

Section B3.7 provides the charging language for Chapter L on design for serviceability.

8. Design for Ponding

As used in this Specification, ponding refers to the retention of water due solely to the deflection of flat roof framing. The amount of this water is dependent on the flexibility of the framing. Lacking sufficient framing stiffness, the accumulated weight of the water can result in the collapse of the roof. The problem becomes catastrophic when more water causes more deflection, resulting in more room for more water until the roof collapses. Detailed provisions for determining ponding stability and strength are given in Appendix 2.

9. Design for Fatigue

Section B3.9 provides the charging language for Appendix 3 on design for fatigue.

10. Design for Fire Conditions

Section B3.10 provides the charging language for Appendix 4 on structural design for fire resistance. Qualification testing is an acceptable alternative to design by analysis for providing fire resistance. It is anticipated that the basis will be ASCE/SFPE Standard 28 (ASCE, 1999), ASTM Standard E119 (ASTM, 2000), and similar documents.

11. Design for Corrosion Effects

Steel members may deteriorate in particular service environments. This deterioration may appear either in external corrosion, which would be visible upon inspection, or in undetected changes that would reduce its strength. The designer should recognize these problems by either factoring a specific amount of tolerance for damage into the design or providing adequate protection systems (for example, coatings, cathodic protection) and/or planned maintenance programs so that such problems do not occur.

Because the interior of an HSS is difficult to inspect, some concern has been expressed regarding internal corrosion. However, good design practice can eliminate the concern and the need for expensive protection. Corrosion occurs in the presence of oxygen and water. In an enclosed building, it is improbable that there would be sufficient reintroduction of moisture to cause severe corrosion. Therefore, internal corrosion protection is a consideration only in HSS exposed to weather.

In a sealed HSS, internal corrosion cannot progress beyond the point where the oxygen or moisture necessary for chemical oxidation is consumed (AISI, 1970). The oxidation depth is insignificant when the corrosion process must stop, even when a corrosive atmosphere exists at the time of sealing. If fine openings exist at connections, moisture and air can enter the HSS through capillary action or by aspiration due to the partial vacuum that is created if the HSS is cooled rapidly (Blodgett, 1967). This can be prevented by providing pressure-equalizing holes in locations that make it impossible for water to flow into the HSS by gravity.

Situations where conservative practice would recommend an internal protective coating include: (1) open HSS where changes in the air volume by ventilation or direct flow of water is possible; and (2) open HSS subject to a temperature gradient that would cause condensation.

HSS that are filled or partially filled with concrete should not be sealed. In the event of fire, water in the concrete will vaporize and may create pressure sufficient to burst a sealed HSS. Care should be taken to keep water from remaining in the HSS during or after construction, since the expansion caused by freezing can create pressure that is sufficient to burst an HSS.

Galvanized HSS assemblies should not be completely sealed because rapid pressure changes during the galvanizing process tend to burst sealed assemblies.

12. Design Wall Thickness for HSS

ASTM A500 tolerances allow for a wall thickness that is not greater than ± 10 percent of the nominal value. Because the plate and strip from which electric-resistance-welded (ERW) HSS are made are produced to a much smaller thickness tolerance, manufacturers in the United States consistently produce ERW HSS with a wall thickness that is near the lower-bound wall thickness limit. Consequently, AISC and the Steel Tube Institute of North America (STI) recommend that 0.93 times the nominal wall thickness be used for calculations involving engineering design properties of ERW HSS. This results in a weight (mass) variation that is similar to that found in other structural shapes. Submerged-arc-welded (SAW) HSS are produced with a wall thickness that is near the nominal thickness and require no such reduction. The design wall thickness and section properties based upon this thickness have been tabulated in AISC and STI publications since 1997.

B4. CLASSIFICATION OF SECTIONS FOR LOCAL BUCKLING

For the purposes of this Specification, steel sections are divided into compact sections, noncompact sections and slender-element sections. Compact sections are capable of developing a fully plastic stress distribution and they possess a rotation capacity of approximately 3 before the onset of local buckling (Yura, Galambos, and Ravindra, 1978). Noncompact sections can develop partial yielding in compression elements before local buckling occurs, but will not resist inelastic local buckling at the strain levels required for a fully plastic stress distribution. Slender-element sections have one or more compression elements that will buckle elastically before the yield stress is achieved.

Limiting Width-Thickness Ratios. The dividing line between compact and noncompact sections is the limiting width-thickness ratio λ_p. For a section to be compact, all of its compression elements must have width-thickness ratios equal to or smaller than the limiting λ_p.

A second limiting width-thickness ratio is λ_r, representing the dividing line between noncompact sections and slender-element sections. As long as the width-thickness ratio of a compression element does not exceed the limiting value λ_r, elastic local buckling will not govern its strength. However, for those cases where the width-thickness ratios exceed λ_r, elastic buckling strength must be considered. Design procedures for such slender-element compression sections are given in Section E7 for members under pure axial compression, and in Sections F3.2, F5.3, F6.2, F7.2, F8.2, F9.3 and F10.3 for beams with a cross section that contains slender plate elements.

The values of the limiting ratios λ_p and λ_r specified in Table B4.1 are similar to those in the 1989 *Specification* (AISC, 1989) and Table 2.3.3.3 of Galambos (1978), except that $\lambda_p = 0.38\sqrt{E/F_y}$, limited in Galambos (1978) to indeterminate beams when moments are determined by elastic analysis and to determinate beams, was adopted for all conditions on the basis of Yura and others (1978).

For greater inelastic rotation capacities than provided by the limiting values λ_p given in Table B4.1, for structures in areas of high seismicity, see Section 8 and Table I-8-1 of the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2005).

Flanges of Built-Up I-Shaped Sections. For built-up I-shaped sections under axial compression (Case 4 in Table B4.1), modifications have been made to the flange local buckling criterion to include web-flange interaction. The k_c in the λ_r *limit* and in Equations E7-7 through E7-9 is the same that is used for flexural members in Equations F3-2 and F5-9. Theory indicates that the web-flange interaction in axial compression is at least as severe as in flexure. Rolled shapes are excluded from this provision because there are no standard sections with proportions where the interaction would occur. In built-up sections where the interaction causes a reduction in the flange local buckling strength, it is likely that the web is also a thin stiffened element.

The k_c factor accounts for the interaction of flange and web local buckling demonstrated in experiments reported in Johnson (1985). The maximum limit of 0.76 corresponds to $F_{cr} = 0.69E/\lambda^2$ which was used as the local buckling strength in editions of both the ASD and LRFD Specifications. An $h/t_w = 27.5$ is required to reach $k_c = 0.76$. Fully fixed restraint for an unstiffened compression element corresponds to $k_c = 1.3$ while zero restraint gives $k_c = 0.42$. Because of web-flange interactions it is possible to get $k_c < 0.42$ from the new k_c formula. If $h/t_w > 5.70\sqrt{E/F_y}$ use $h/t_w = 5.70\sqrt{E/F_y}$ in the k_c equation, which corresponds to the 0.35 limit.

Webs in Flexure. New formulas for λ_p are presented in Case 11 in Table B4.1 for I-shaped beams with unequal flanges. These provisions are based on research reported in White (2003).

Rectangular HSS in Compression. The limits for rectangular HSS walls in uniform compression (Case 12 in Table B4.1) have been used in AISC Specifications since 1969. They are based on Winter (1968), where adjacent stiffened compression elements in box sections of uniform thickness were observed to provide negligible torsional restraint for one another along their corner edges. The λ_p limit for plastic analysis is adopted from *Limit States Design of Steel Structures* (CSA, 1994). The web slenderness limits are the same as those used for webs in wide-flange shapes.

Lower values of λ_p are specified for high-seismic design in the *Seismic Provisions for Structural Steel Buildings* based upon tests (Lui and Goel, 1987) that have shown that rectangular HSS braces subjected to reversed axial load fracture catastrophically under relatively few cycles if a local buckle forms. This was confirmed in tests (Sherman, 1995) where rectangular HSS braces sustained over 500 cycles when a local buckle did not form, even though general column buckling had occurred, but failed in less than 40 cycles when a local buckle developed. The seismic λ_p is based upon tests (Lui and Goel, 1987) of HSS that had a small enough *b*/*t* ratio so that braces performed satisfactorily for members with reasonable column slenderness. Filling the rectangular HSS with lean concrete (concrete mixed with a low proportion of cement) has been shown to effectively stiffen the HSS walls and improve cyclic performance.

Rectangular HSS in Flexure. A significant change from previous editions of the Specification is the compactness limit for webs in rectangular HSS flexural members (Case 13 in Table B4.1). The previously used value of $\lambda_p = 3.76\sqrt{E/F_y}$ was reduced to $\lambda_p = 2.42\sqrt{E/F_y}$. This change was introduced because tests reported in Wilkinson and Hancock (1998 and 2002) showed that HSS beams with geometries at the previous limiting compactness had hardly any rotation capacity available and were thus unable to deliver a target rotation capacity of 3.

Round HSS in Compression. The λ_r limit for round HSS in compression (Case 15 in Table B4.1) was first used in the 1978 *ASD Specification*. It was recommended in Schilling (1965) based upon research reported in Winter (1968). The same limit was also used to define a compact shape in bending in the 1978 *ASD Specification*. However, the limits for λ_p and λ_r were changed in the 1986 *LRFD Specification* based upon experimental research on round HSS in bending (Sherman, 1985;

Galambos, 1998). Excluding the use of round HSS with $D/t > 0.45E/F_y$ was also recommended in Schilling (1965).

Following the SSRC recommendations (Galambos, 1998) and the approach used for other shapes with slender compression elements, a Q factor is used for round sections to account for interaction between local and column buckling in Section E7.2(c). The Q factor is the ratio between the local buckling stress and the yield stress. The local buckling stress for the round section is taken from the AISI provisions based on *inelastic action* (Winter, 1970) and is based on tests conducted on fabricated and manufactured cylinders. Subsequent tests on fabricated cylinders (Galambos, 1998) confirm that this equation is conservative.

Round HSS in Flexure. The high shape factor for round hollow sections (Case 15 in Table B4.1) makes it impractical to use the same slenderness limits to define the regions of behavior for different types of loading. In Table B4.1, the values of λ_p for a compact shape that can achieve the plastic moment, and λ_r for bending, are based on an analysis of test data from several projects involving the bending of round HSS in a region of constant moment (Sherman and Tanavde, 1984; Galambos, 1998). The same analysis produced the equation for the inelastic moment capacity in Section F7. However, a more restrictive value of λ_p is required to prevent inelastic local buckling from limiting the plastic hinge rotation capacity needed to develop a mechanism in a round HSS (Sherman, 1976).

The values of λ_r for axial compression and for bending are both based on test data. The former value has been used in building specifications since 1968 (Winter, 1970). Section F8 also limits the D/t ratio for any round section to $0.45E/F_y$. Beyond this, the local buckling strength decreases rapidly, making it impractical to use these sections in building construction.

B5. FABRICATION, ERECTION AND QUALITY CONTROL

Section B5 provides the charging language for Chapter M on fabrication, erection and quality control.

B6. EVALUATION OF EXISTING STRUCTURES

Section B6 provides the charging language for Appendix 5 on the evaluation of existing structures.

CHAPTER C

STABILITY ANALYSIS AND DESIGN

Chapter C addresses the stability analysis and design requirements for steel buildings and related structures. The chapter has been reorganized from the previous Specifications into two parts: Section C1 outlines general requirements for stability and specific stability requirements for individual members (for example, beams, columns, braces) and for systems, including moment frames, braced frame and shear walls, gravity frame systems, and combined systems. Section C2 addresses the calculation of required strengths including the definition of acceptable analysis methods and specific constraints to be placed on the analysis and design procedures. A discussion of the effective length factor, K, the column buckling stress, F_e, and associated buckling analysis methods is provided at the end of the commentary chapter.

C1. STABILITY DESIGN REQUIREMENTS

1. General Requirements

The stability of structures must be considered from the standpoint of the structure as a whole, including not only the compression members, but also the beams, bracing systems and connections. Stability of individual components must also be provided. Considerable attention has been given in the technical literature to this subject, and various methods are available to provide stability (Galambos, 1998). In all approaches, the method of analysis and the equations for component strengths are inextricably interlinked. Traditionally, the effects of unavoidable geometric imperfections (within fabrication and erection tolerances) and distributed yielding at strength limit states (including residual stress effects) are addressed solely within member strength equations. Correspondingly, structural analysis is conducted using the nominal or undeformed structure geometry and elastic stiffness. This Specification addresses this traditional approach, termed the Effective Length Method in this commentary, as well as a new approach which is termed the Direct Analysis Method, addressed in Appendix 7. The Direct Analysis Method includes nominal geometric imperfection and stiffness reduction effects directly within the structural analysis. In either the Effective Length or the Direct Analysis Method, structural analysis by itself is not sufficient to provide for the stability of the structure as a whole. The overall stability of the structure as well as the stability of individual elements is provided for by the combined calculation of the required strengths by structural analysis and the satisfaction of the member and connection design provisions of this Specification.

In general, it is essential that an accurate second-order analysis of the structure be performed. The analysis should consider the influence of second-order effects (including P-Δ and P-δ effects as shown in Figure C-C1.1) and of flexural, shear and axial deformations. More rigorous analysis methods allow formulations of simpler limit state models. One such example can be found in Appendix 7, where the new Direct Analysis Method is presented as an alternative method to improve and simplify design for stability. In this case, the inclusion of nominal geometric imperfection and member stiffness reduction effects directly in the analysis allows the use of $K = 1.0$ in calculating the in-plane column strength, P_n, within the beam-column interaction equations of Chapter H. This simplification comes about because the Direct Analysis Method provides a better estimate of the true load effects within the structure. The Effective Length Method, in contrast, includes the above effects indirectly within the member strength equations.

2. Member Stability Design Requirements

Chapters E through I contain the necessary provisions for satisfying member stability (in other words, the available strengths) given the load effects obtained from structural analysis and given specific bracing conditions assumed in the calculation of the member strengths. Where beam and column members rely upon braces that are not part of the lateral load resisting system to define their unbraced length, the braces themselves must have sufficient strength and stiffness to control member movement at the brace points. Appendix 6 contains all the requirements for braces that were previously contained within Chapter C of the 1999 *LRFD Specification* (AISC, 2000b). Design requirements for braces that are part of the lateral load resisting system (that is, braces that are included within the analysis of the structure) are addressed within Chapter C.

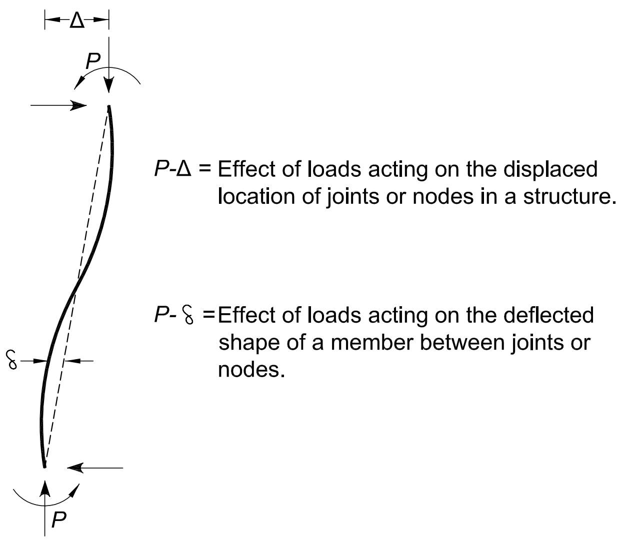


Fig. C-C1.1. P-Δ and P-δ effects in beam-columns.

3. System Stability Design Requirements

Lateral stability can be provided by braced frames, shear-wall systems, moment frames or any other comparable lateral load resisting systems. Where combined systems are used, it is important that consideration be given to the transfer of forces and load sharing between systems, and to the destabilizing effect of vertical load carrying elements not participating as part of the lateral load resisting system (for example, leaning columns).

3a. Braced-Frame and Shear-Wall Systems

Braced-frame systems are commonly analyzed and designed as vertically cantilevered pin-connected truss systems, ignoring any secondary moments within the system. The effective length factor, K, of components of the braced frame is normally taken to be 1.0, unless a smaller value is justified by structural analysis and the member and connection design is consistent with this assumption. Use of a K-factor less than 1.0 is discussed further at the end of this commentary chapter.

3b. Moment-Frame Systems

Moment-frame systems rely primarily upon the flexural stiffness of the connected beams and columns although the reduction in the stiffness due to shear deformations can be important and should be considered where column bays are short and/or members are deep. Except as noted in Section C2.2a(4), Section C2.2b and Appendix 7, the design of all columns and beam-columns must be based on an effective length, KL, greater than the actual length determined as specified in Section C2. The Direct Analysis Method in Appendix 7, as well as the provisions of Sections C2.2a(4) and C2.2b, provide the means for proportioning columns with $K = 1.0$.

3c. Gravity Framing Systems

Columns in gravity framing systems can be designed as pin-ended columns with $K = 1.0$. However, the destabilizing effect (P-Δ effect) of the gravity load on all such columns and the load transfer from these columns to the lateral load resisting system must be accounted for in the design of the lateral load resisting system. Methods for including this leaning column effect in the design of the lateral system are discussed in Commentary Section C2.

3d. Combined Systems

When combined systems are used, structural analysis must proportion the lateral loads to the various systems with due regard to the relative stiffness of each system and the load transfer path between them. Consideration must be given to the variation in stiffness inherent in concrete or masonry shear walls due to various degrees of cracking possible. This applies both to serviceability load combinations and strength load combinations. It is prudent for the designer to consider a range of possible stiffnesses, with due regard to shrinkage, creep and load history, in order to envelope the likely behavior and provide sufficient strength in all interconnecting elements between systems. Once the loads are determined on each system, the design must conform to all requirements for the respective systems.

C2. CALCULATION OF REQUIRED STRENGTHS

This Specification recognizes a variety of analysis and design procedures for assessing the response of lateral load resisting systems. These include the use of second-order inelastic and plastic methods with specially developed computer software, effective length factors in conjunction with second-order elastic analysis, the Direct Analysis Method, and simplified first-order elastic methods suitable for manual calculation. Accordingly, Section C2 addresses several general analysis approaches commonly used and defines certain constraints that must be placed on the analysis and design with each method so as to provide a safe design.

1. Methods of Second-Order Analysis

Some of the key differences between the 1999 *LRFD Specification* (AISC, 2000b) and this Specification involve requirements for minimum stiffness and strength of steel frames. The provisions in AISC (2000b) imposed the following two requirements on braced frames only:

(1) A minimum brace strength of

$$P_{br} = 0.004\Sigma P_u$$

(2) A minimum brace stiffness of

$$\beta_{br} = 2\Sigma P_u/(\phi L) \text{ where } \phi = 0.75$$

By substituting the minimum required brace stiffness, β_{br}, into the B_2 equation below [Equation C1-4 in AISC (2000b) where $\beta_{br} = \Sigma H/\Delta_{oh}$], it can be observed that the above minimum brace stiffness is equivalent to providing $B_2 \leq 1.6$. The minimum brace force, $P_{br} = 0.004\,\Sigma P_u$, is the force one would obtain in the brace by doing a first-order elastic analysis at the strength load level, including an initial out-of-plumbness of 0.002 times the story height, L, and assuming an amplification from second-order effects of 2.0. The amplification of 2.0 is determined using $\beta_{br} = 2\Sigma P_u/(\phi L)$ in the B_2 equation below, but without including the ϕ factor on stiffness.

$$B_2 = \frac{1}{1 - \dfrac{\Sigma P_u \Delta_{oh}}{\Sigma HL}} = \frac{1}{1 - \dfrac{\Sigma P_u}{\beta_{br} L}} \tag{C-C2-1}$$

In contrast, this Specification imposes a minimum stiffness on all frames by application of a B_2 limit of 1.5 unless the more accurate Direct Analysis Method of Appendix 7 is used. The Direct Analysis Method addresses the influence of nominal geometric imperfections (for example, out-of-plumbness) and stiffness reductions due to distributed yielding directly within the analysis, in which case the above stiffness and strength requirements are accounted for in a direct manner. Setting the B_2 equation above to 1.5 is equivalent to imposing a minimum frame stiffness of $\beta_{br} = 3\ \Sigma P_u/L$ which is 12 percent larger than in AISC (2000b) for braced systems. The 12 percent difference is a consequence of setting the B_2 limit at 1.5 for all frames designed without the use of the more accurate Direct Analysis Method. Additional discussion about upper limits on B_2 can be found in Appendix 7, Section 7.3.

In the development of this Specification, it was considered to require an additive notional load of $0.002\Sigma Y_i$ with all load combinations for all B_2 levels. However, $(\Sigma H + 0.002\Sigma Y_i)/\Sigma H$ is close to 1.0 for all of the lateral load combinations in SEI/ASCE 7 (ASCE, 2002), and for $B_2 \leq 1.5$, the additional internal forces caused by applying $0.002\Sigma Y_i$ in combination with the required lateral loadings are small and may be neglected. Therefore, $0.002\Sigma Y_i$ is required only as a minimum lateral load in the gravity load-only combinations within Section C2.2a. Conversely, for frames with $B_2 > 1.5$, the P-Δ effects associated with the amplified lateral deflections due to initial out-of-plumbness plus the additional amplified defections due to distributed yielding or other incidental causes can be significant at strength load levels. Therefore, for these stability-sensitive structures the Direct Analysis Method of Appendix 7 is required with the use of an additive notional lateral load of $N_i = 0.002\Sigma Y_i$.

1a. General Second-Order Elastic Analysis

Section C2.1a states that any second-order elastic analysis method that captures both the P-Δ and P-δ effects, when one or both are significant to the accurate determination of internal forces, may be used. The amplification of first-order analysis forces by the traditional B_1 and B_2 factors as defined in Section C2.1b is one method of conducting an approximate second-order elastic analysis. In addition, the section states that all flexural, shear and axial deformations that significantly affect the stability of the structure and its elements in general must be considered. Also, in the Direct Analysis Method, nominal geometric imperfections and member stiffness reduction due to residual stresses must be directly included in the analysis.

The Direct Analysis Method is more sensitive to the accuracy of the second-order elastic analysis than the Effective Length Method. The Direct Analysis method may be used in the analysis and design of all lateral load resisting systems. The Commentary to Appendix 7, Sections 7.1 and 7.3, contains specific guidelines on the requirements for rigorous second-order elastic analysis, and provides benchmark problems that may be used to determine the adequacy of a particular analysis method. Software programs being used in the analysis should be tested with these benchmark problems to check their accuracy and to understand their limitations. Also, it is essential for the designer to apply the specific constraints applicable to the analysis-design method being used.

It is important to recognize that traditional elastic analysis methods, even those that properly consider second-order effects, are based on the undeformed geometry and nominal member properties and stiffnesses. Initial imperfections in the structure, such as out-of-plumbness, fabrication tolerances, incidental patterned gravity loading, temperature gradients across the structure, foundation settlements, etc., as well as residual member stresses and general softening of the structure at the strength limit state, combine with the destabilizing effects of the vertical loads to increase the magnitude of load effects in the structure above those predicted by traditional analysis methods. This is particularly true for stability-sensitive structures containing large vertical loads with small lateral load requirements, leading to relatively low lateral load resistance. Limits on B_2 are placed on some of the analysis-design methods to limit the potential underestimation of load effects in stability-sensitive structures. Note that B_2 may be determined directly as the ratio of the second-order to the first-order lateral displacements at each story in the structure, $\Delta_{2nd\ order}/\Delta_{1st\ order}$, (the appropriate definition when a second-order analysis is performed), or as defined by Equation C2-3 (the appropriate definition when an amplified first-order analysis is performed). This underestimation of load effects is particularly important in the design of restraining girders of moment frames and braces in braced frames. Within the Effective Length Method, the in-plane column strength, P_n, accounts for the above effects by inclusion of the effective length factor and the use of the column strength curve of Section E3. However, the increases in the magnitude of the internal forces due to these effects are not accounted for within other member and connection design equations. The Direct Analysis Method in Appendix 7 overcomes these shortcomings in the traditional Effective Length Method. Therefore, it is recommended for use, particularly in stability-sensitive structures.

1b. Second-Order Analysis by Amplified First-Order Elastic Analysis

Section C2.1b addresses the traditional amplified first-order analysis method that has long been part of this Specification. It has been expanded for use in systems where axial load is predominant, such as braced frames and truss systems, as well as moment frames. Where properly applied, this method constitutes an acceptable elastic second-order analysis method.

This first-order analysis method defines amplification factors B_1 and B_2 that are applied to the first-order forces so as to obtain an estimate of the second-order forces. In the general case, a member may have first-order load effects not associated with sidesway that are multiplied by B_1 and first-order load effects produced by sidesway that are multiplied by B_2. The factor B_1 is required to estimate the P-δ effects on the nonsway moments, M_{nt}, in axially loaded members, while the factor B_2 is required to estimate the P-Δ effect in frame components of braced, moment and/or combined framing systems. The P-Δ and P-δ effects are shown graphically in Figure C-C1.1 for a beam column. The effect of B_1 and B_2 amplification of moments is shown in Figure C-C2.1.

The factor B_2 applies only to internal forces associated with sidesway and is calculated for an entire story. In building frames designed to limit Δ_H/L to a predetermined value, the factor B_2 may be found in advance of designing individual members by using the target maximum limit on Δ_H/L within Equation C2-6b.

In determining B_2 and the second-order effects on the lateral load resisting system, it is important that Δ_H include not only the interstory displacement in the plane of the lateral load resisting system, but also any additional displacement in the floor or roof diaphragm or horizontal framing system that may increase the overturning effect of columns attached to and "leaning" against the horizontal system. Either the maximum displacement or a weighted average displacement, weighted in proportion to column load, should be considered.

Drift limits may also be set for design of various categories of buildings so that the effect of secondary bending is reduced (ATC, 1978; Kanchanalai and Lu, 1979). However, drift limits alone are not sufficient to allow stability effects to be neglected (LeMessurier, 1977).

Both types of first-order moments, M_{nt} and M_{lt}, may be induced by gravity loads. M_{nt} is defined as a moment developed in a member with frame sidesway prevented. M_{lt} is the moment developed within a member due to frame sidesway. If a significant restraining force is necessary to prevent sidesway of an unsymmetrical structure or an unsymmetrically loaded symmetrical structure, the moments induced by releasing the restraining force contribute to the M_{lt} moments. In most reasonably symmetric frames, this effect will be small. If the moment B_2M_{lt} is added algebraically to the B_1M_{nt} moment developed with sidesway prevented, as defined by Equation C2-1a, a reasonably accurate value of M_r results in most cases. A rigorous second-order elastic analysis is recommended for accurate determination of the frame internal forces when B_1 is larger than about 1.2. End moments produced in sidesway frames by lateral loads from wind or earthquake are always M_{lt} moments. Note that, in general, axial forces must also be amplified

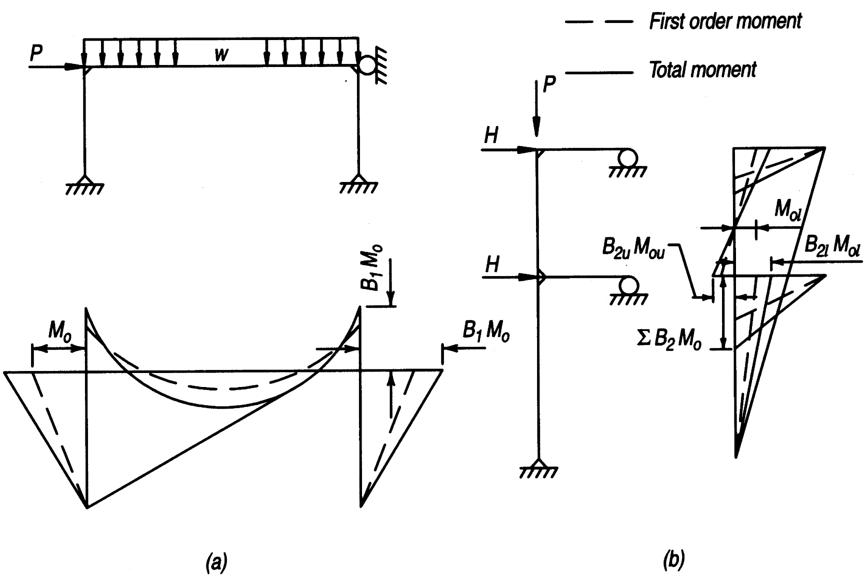


Fig. C-C2.1. Moment amplification.

according to Equation C2-1b for braced and moment frames, although the effect may be small in many low rise moment frames.

When first-order end moments in members subject to axial compression are magnified by B_1 and B_2 factors, equilibrium requires that they be balanced by moments in connected members (for example, see Figure C-C2.1). The associated second-order internal moments in the connected members can be calculated satisfactorily in most cases by amplifying the moments in all the members of the lateral load resisting system, in other words, the columns and the beams, by their corresponding B_1 and B_2 values. For beam members, the larger of the B_2 values from the story above or below is used. Connections shall also be designed to resist the magnified end moments. Alternatively, the difference between the magnified moment and the first-order moment in the column(s) at a given joint may be distributed to any other moment-resisting members attached to the compressed member (or members) in proportion to the relative stiffness of the uncompressed members. Minor imbalances may be neglected in the judgment of the engineer. This latter method is considerably more tedious than the above recommended method. Complex conditions, such as occur when there is significant magnification in several members meeting at a joint, may require an actual second-order elastic analysis rather than an amplified first-order analysis.

In braced and moment frames, P_n is governed by the maximum slenderness ratio regardless of the plane of bending, if the member is subject to significant biaxial bending, or if Section H1.3 is not utilized. Section H1.3 is an alternative approach for checking beam-column strength that provides for the separate checking of beam-column in-plane and out-of-plane stability in members predominantly subject to bending within the plane of the frame. However, P_{e1} and P_{e2} expressed by Equations C2-5 and C2-6a are always calculated using the slenderness ratio in the plane of bending. Thus, when flexure in a beam-column is about the strong axis only, two different values of slenderness ratio may be involved in the amplified first-order elastic and design calculations.

The value of $R_M = 0.85$ within Equation C2-6b is based on an approximate upper-bound influence of P-δ effects on the amplification of the sidesway displacements in practical moment frames (LeMessurier, 1977).

The second-order internal forces from separate structural analyses cannot normally be combined by superposition since second-order amplification depends, in a nonlinear fashion, on the total axial forces within the structure. Therefore, a separate second-order analysis must be conducted for each load combination considered in the design. The first-order internal forces, calculated prior to amplification within the amplified first-order elastic analysis procedure of Section C2.1b, may be superimposed to determine the total first-order internal forces.

When bending occurs about both the x- and the y-axes, the required flexural strength, calculated about each axis, is amplified by B_1 based on the value of C_m and P_{e1} in Equation C2-2 corresponding to the moment gradient in the beam-column and its slenderness ratio in the plane of bending. A similar amplification by B_2 in the required flexural strength must occur for ΣP_{e2} in Equation C2-3 corresponding to the in-plane response.

Equations C2-2 and C2-4 are used to approximate the maximum second-order moments in compression members with no relative joint translation and no transverse loads between the ends of the member. Figure C-C2.2a compares the approximation for C_m in Equation C2-4 to the exact theoretical solution for beam-columns subjected to applied end moments (Chen and Lui, 1987). This figure plots the approximate and analytical values of C_m versus the end-moment ratio M_1/M_2 for several levels of $P/P_e(P_e = P_{e1}$ with $K = 1)$. Figure C-C2.2b shows the corresponding approximate and analytical solutions for the maximum second-order elastic moment within the member, M_r, versus the axial load level, P/P_e, for several values of the end moment ratio M_1/M_2.

For beam-columns with transverse loadings, the second-order moment can be approximated by

$$C_m = 1 + \psi\left(\frac{\alpha P_r}{P_{e1}}\right) \tag{C-C2-2}$$

for simply supported members

where

$\psi = \dfrac{\pi^2 \delta_o EI}{M_o L^2} - 1$

δ_o = maximum deflection due to transverse loading, in. (mm)

M_o = maximum first-order moment within the member due to the transverse loading, kip-in. (N-mm)

α = 1.0 (LRFD) or 1.6 (ASD)

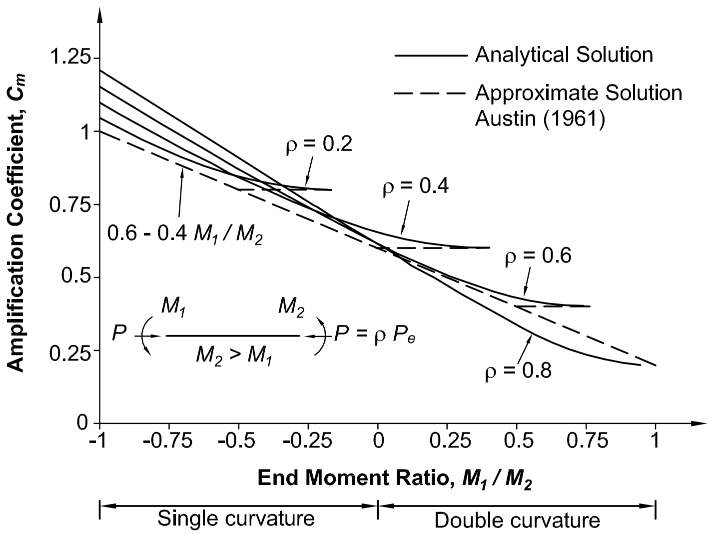


Fig. C-C2.2a. Equivalent moment factor C_m for beam-columns subjected to applied end moments.

For restrained ends, some limiting cases are given in Table C-C2.1 together with two cases of simply supported beam-columns (Iwankiw, 1984). These values of C_m are always used with the maximum moment in the member. For the restrained-end cases, the values of B_1 are most accurate if values of $K < 1.0$, corresponding to the member end conditions, are used in calculating P_{e1}. In lieu of using the equations above, $C_m = 1.0$ is used conservatively for all transversely loaded members. It can be shown that the use of $C_m = 0.85$ for members with restrained ends, specified in previous Specifications, can sometimes result in a significant under-estimation of the internal moments. Therefore, the use of $C_m = 1.0$ is recommended as a simple conservative approximation for all cases involving transversely loaded members.

2. Design Requirements

Section C2.2 contains requirements for two of the three methods of elastic analysis and design of lateral load resisting frames allowed by this Specification: (a) design by elastic second-order analysis; and (b) design by elastic first-order analysis. Conformance to all the constraints of these methods as specified in this section satisfies the requirements of Section C1.1. Appendix 7 addresses the third method of analysis and design called the Direct Analysis Method. Both methods listed in this section specify that the structure should be analyzed using the nominal geometry and the nominal elastic stiffnesses (*EI, EA*) for all members, which is the traditional approach. In order to limit potential errors in the load effects in the structure from these simplified analyses, it is necessary to limit the sidesway amplification, as represented by $\Delta_{2nd\ order}/\Delta_{1st\ order}$ (or equivalently, the

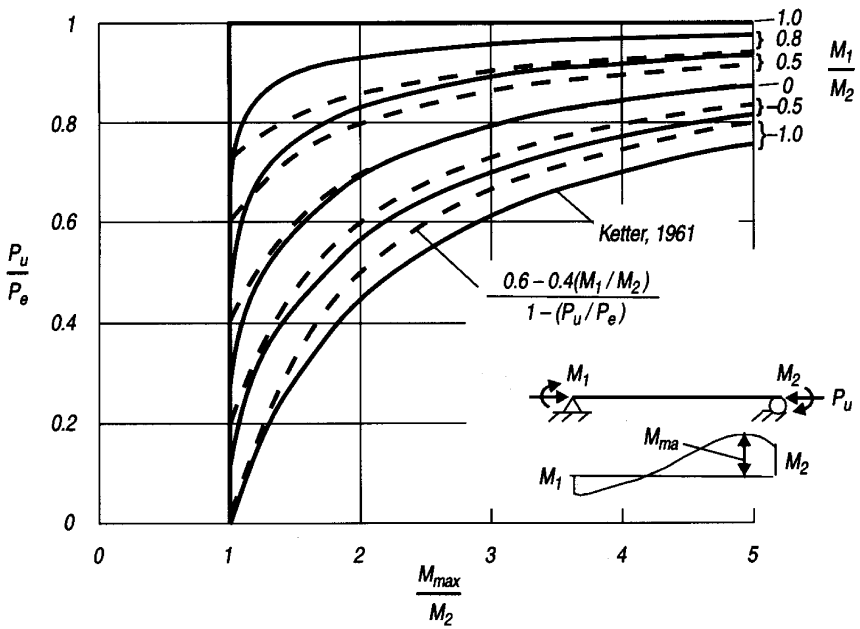
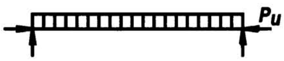
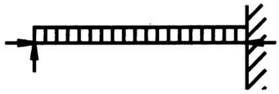
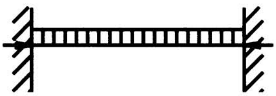
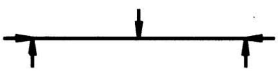
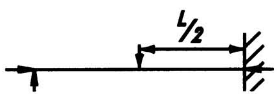
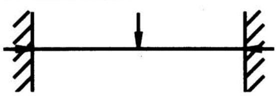


Fig. C-C2.2b. Second-order moments for beam-columns subjected to applied end moments.

TABLE C-C2.1
Amplification Factors ψ and C_m

Case	Ψ	C_m
	0	1.0
	–0.4	$1-0.4\dfrac{P_u}{P_{e1}}$
	–0.4	$1-0.4\dfrac{P_u}{P_{e1}}$
	–0.2	$1-0.2\dfrac{P_u}{P_{e1}}$
	–0.3	$1-0.3\dfrac{P_u}{P_{e1}}$
	–0.2	$1-0.2\dfrac{P_u}{P_{e1}}$

B_2 amplifier), in each story of the frame for all load combinations. A limit of 1.5 on $\Delta_{2nd\ order}/\Delta_{1st\ order}$ is specified for each of the methods addressed in Section C2.2a and C2.2b. Otherwise, the Direct Analysis Method in Appendix 7 is required. The Direct Analysis Method is applicable for any building frame, regardless of the sidesway amplification or B_2 value, and its use is encouraged.

It is important to note that the sidesway amplification or B_2 limits specified in Chapter C and Appendix 7 are based on Equation C2-3 which specifies a first-order elastic analysis using the nominal geometry and properties of the structure.

2a. Design by Second-Order Analysis

It is essential that the analysis of the frame be carried out at the strength limit state because of the nonlinearity associated with second-order effects. For design by the ASD method, this load level is estimated to be 1.6 times the ASD load combinations. This requirement is specified in clause (2).

Clause (3) in this section requires that, for all gravity load only combinations, a minimum lateral load of $0.002Y_i$ shall be applied at each level of the structure, where Y_i is the design gravity load acting on level i. Note that the load is to be applied independently in two orthogonal directions on the structure. Note also that the column strengths, P_n, in moment frames must be based on the effective buckling length, KL, or the column buckling stress, F_e, where either KL or F_e is determined from a sidesway buckling analysis of the structure. A detailed discussion of the K-factor, the column buckling stress, F_e, and associated sidesway buckling analysis methods is provided at the end of this commentary chapter.

In the special case where the sidesway amplification $\Delta_{2nd\ order}/\Delta_{1st\ order}$ (or B_2) ≤ 1.1, the frame design may be based on the use of $K = 1.0$ for the columns, as specified in clause (4). By limiting the sidesway amplification (or B_2 level) to a maximum value of 1.1, the resulting unconservative error is limited to a maximum of approximately 6 percent within the in-plane beam-column strength checks of Chapter H (White and Hajjar, 1997).

For all cases, braced frames may be designed on the basis of $K = 1.0$.

2b. Design by First-Order Analysis

This section provides a method for designing frames using a first-order elastic analysis with $K = 1.0$, provided the sidesway amplification $\Delta_{2nd\ order}/\Delta_{1st\ order} \leq 1.5$ (or $B_2 \leq 1.5$, where B_2 is determined as specified within the amplified first-order elastic analysis procedure of Section C2.1) and the required compressive strength of all members that are part of the lateral load resisting frame (other than truss members whose flexural stiffness is neglected in the analysis) have $\alpha P_r < 0.5 P_y$. All load combinations must include an additional lateral load, N_i, applied in combination with other loads at each level of the structure specified by Equation C2-8. Note that the load is to be applied independently in two orthogonal directions on the structure. If drift occurs under gravity load, then the minimum load should be applied in the direction of the drift. This equation is derived from the Direct Analysis Method as shown in the commentary to Appendix 7. It is based on an assumed $\Delta_{2nd\ order}/\Delta_{1st\ order}$ (or B_2) value of 1.5. Initial out-of-plumbness does not need to be considered in the calculation of Δ. Equation C2-8 is based on the clause within Appendix 7 that permits the notional load to be applied as a minimum lateral load in the *gravity load only* combinations and not in combination with other lateral loads when $\Delta_{2nd\ order}/\Delta_{1st\ order}$ (or B_2) ≤ 1.5. The minimum value of N_i of $0.0042Y_i$ is based on the assumption of a minimum first-order drift ratio due to any effects of $\Delta/L = 0.002$. Note that a target maximum drift ratio, corresponding to drifts under either the LRFD strength load combinations or 1.6 times the ASD strength load combinations, can be assumed at the start of design to determine the additional lateral load N_i. As long as that drift ratio is not exceeded at any strength load level, the design will be conservative.

The nonsway amplification of beam-column moments is addressed within the procedure specified in this section by applying the B_1 amplifier of Section C2.1 conservatively to the total member moments. In many cases involving beam-columns not subject to transverse loading between supports in the plane of bending, $B_1 = 1.0$.

Further explanation of this first-order design procedure is provided at the end of Appendix 7.

Determination of Effective Length Factor, *K*, or the Column Buckling Stress, F_e

There are two uses for the effective length factor, K, within the Specification:

(1) *Amplified first-order analysis.* K is used in the determination of the elastic buckling load, P_{e1}, for a member, or ΣP_{e2} for a building story, for calculation of the corresponding amplification factors B_1 and B_2 within the amplified first-order elastic analysis procedure of Section C2.1b; and

(2) *Column flexural buckling strength, P_n.* K is used in the determination of the column flexural buckling strength, P_n, from Chapter E, which may be based either on elastic or inelastic buckling analysis.

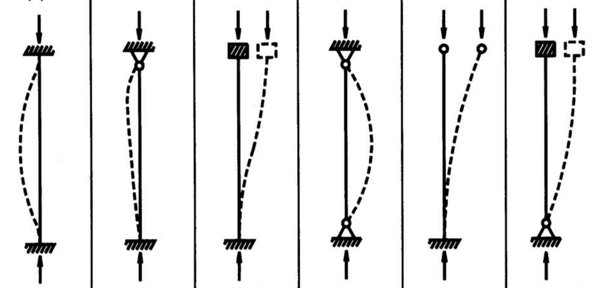
Each of these uses is discussed in detail below. The section begins, however, with a discussion of some background on the effective length factor, K, and some traditional approaches to determine K, most notably from the alignment charts.

Traditional Approaches to Calculating K—The Alignment Charts. A wide range of methods have been suggested in the engineering literature for the calculation of column effective length factors, K (Kavanagh, 1962; Johnston, 1976; LeMessurier, 1977; ASCE Task Committee on Effective Length, 1997; White and Hajjar, 1997a). These range from simple idealizations of single columns such as shown in Table C-C2.2 to complex buckling solutions for specific frames and loading conditions. In some types of frames, K-factors are easily estimated or calculated and they serve as a convenient tool for stability design. In other types of structures, the determination of accurate K-factors is tedious by hand procedures, and system stability may be assessed more effectively without the consideration of member K values at all. This latter approach is addressed in more detail later in this section.

The most common method for determining K is with the use of the alignment charts, also commonly referred to as the nomographs, shown in Figure C-C2.3 for frames with sidesway inhibited and Figure C-C2.4 for frames with sidesway uninhibited. (Kavanagh, 1962) The appropriate subassemblages upon which the charts are based are shown in the figure, along with the alignment chart. The alignment charts are based on assumptions of idealized conditions which seldom exist in real structures. These assumptions are as follows:

1. Behavior is purely elastic
2. All members have constant cross section.
3. All joints are rigid.
4. For columns in frames with sidesway inhibited, rotations at opposite ends of the restraining beams are equal in magnitude and opposite in direction, producing single curvature bending.
5. For columns in frames with sidesway uninhibited, rotations at opposite ends of the restraining beams are equal in magnitude and direction, producing reverse curvature bending.

TABLE C-C2.2
Approximate Values of Effective Length Factor, *K*

Buckled shape of column is shown by dashed line.	(a)	(b)	(c)	(d)	(e)	(f)
						
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.10	2.0
End condition code	 *Rotation fixed and translation fixed* *Rotation free and translation fixed* *Rotation fixed and translation free* *Rotation free and translation free*					

6. The stiffness parameter $L\sqrt{P/EI}$ of all columns is equal.
7. Joint restraint is distributed to the column above and below the joint in proportion to EI/L for the two columns.
8. All columns buckle simultaneously.
9. No significant axial compression force exists in the girders.

The alignment chart for sidesway inhibited frames shown in Figure C-C2.3 is based on the following equation:

$$\frac{G_A G_B}{4}(\pi/K)^2 + \left(\frac{G_A + G_B}{2}\right)\left(1 - \frac{\pi/K}{\tan(\pi/K)}\right) + \frac{2\tan(\pi/2K)}{(\pi/K)} - 1 = 0$$

The alignment chart for sidesway uninhibited frames shown in Figure C-C2.4 is based on the following equation:

$$\frac{G_A G_B(\pi/K)^2 - 36}{6(G_A + G_B)} - \frac{(\pi/K)}{\tan(\pi/K)} = 0$$

where

$$G = \frac{\Sigma(E_c I_c/L_c)}{\Sigma\left(E_g I_g/L_g\right)} = \frac{\Sigma(EI/L)_c}{\Sigma(EI/L)_g}$$

The subscripts A and B refer to the joints at the ends of the column being considered. The symbol Σ indicates a summation of all members rigidly connected to that joint and lying in the plane in which buckling of the column is being considered. E_c is the modulus of the column, I_c is the moment of inertia of the column, and L_c is the unsupported length of the column. E_g is the modulus of the girder, I_g is the moment of inertia of the girder, and L_g is the unsupported length of the girder or other restraining member. I_c and I_g are taken about axes perpendicular to the plane of buckling being considered. The alignment chart is valid for different materials if an appropriate effective rigidity, EI, is used in the calculation of G.

For column ends supported by, but not rigidly connected to, a footing or foundation, G is theoretically infinity but unless designed as a true friction-free pin, may be taken as 10 for practical designs. If the column end is rigidly attached to a properly designed footing, G may be taken as 1.0. Smaller values may be used if justified by analysis.

Theoretical K values obtained from the alignment charts for various idealized end conditions, rotation fixed or free and translation fixed or free, are shown in Table C-C2.2 along with practical recommendations for use in actual design.

It is important to remember that the alignment charts are based on the assumptions of idealized conditions previously discussed and that these conditions seldom exist

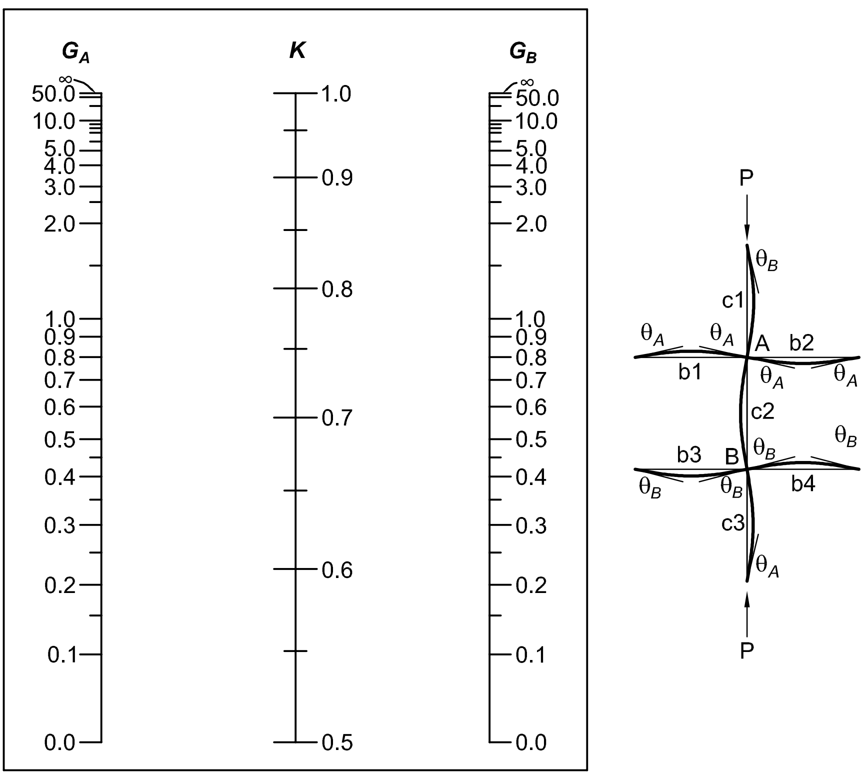


Fig. C-C2.3. Alignment chart—sidesway inhibited (braced frame).

in real structures. Therefore, adjustments are required when these assumptions are violated and the alignment charts are still to be used. Adjustments for common design conditions that apply to both sidesway conditions are:

1. To account for inelasticity in columns, replace $(E_c I_c)$ with $\tau_a(E_c I_c)$ for all columns in the expression for G_A and G_B. The stiffness reduction factor, τ_a, is discussed later in this section.
2. For girders containing significant axial load, multiply the $(EI/L)_g$ by the factor $(1-Q/Q_{cr})$ where Q is the axial load in the girder and Q_{cr} is the in-plane buckling load of the girder based on $K = 1.0$.

For sidesway inhibited frames, these adjustments for different beam end conditions may be made:

1. If the far end of a girder is fixed, multiply the $(EI/L)_g$ of the member by 2.0.
2. If the far end of the girder is pinned, multiply the $(EI/L)_g$ of the member by 1.5.

For sidesway uninhibited frames and girders with different boundary conditions, the modified girder length, L'_g, should be used in place of the actual girder length,

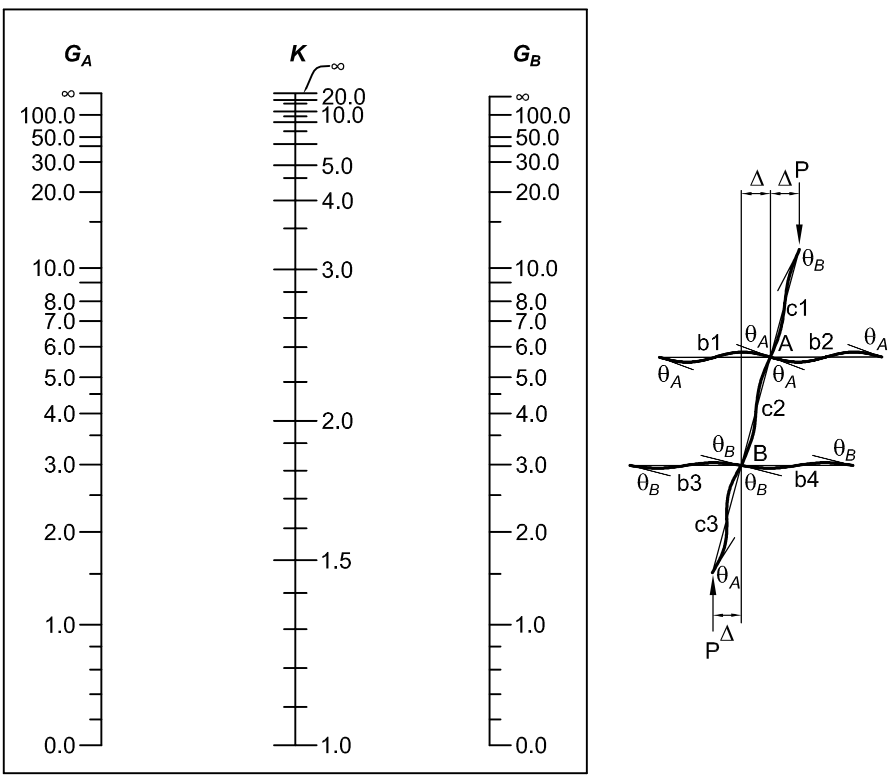


Fig. C-C2.4. Alignment chart—sidesway uninhibited (moment frame).

where

$$L'_g = L_g\,(2 - M_F/M_N)$$

M_F is the far end girder moment and M_N is the near end girder moment from a first-order lateral analysis of the frame. The ratio of the two moments is positive if the girder is in reverse curvature. If M_F/M_N is more than 2.0, then L'_g becomes negative, in which case G is negative and the alignment chart equation must be used.

1. If the far end of a girder is fixed, multiply the $(EI/L)_g$ of the member by $2/3$.
2. If the far end of the girder is pinned, multiply the $(EI/L)_g$ of the member by 0.5.

One important assumption in the development of the alignment charts is that all beam-column connections are fully restrained (FR connections). As seen above, when the far end of a beam does not have an FR connection that behaves as assumed, an adjustment must be made. When a beam connection at the column under consideration is a shear only connection—that is, there is no moment—then that beam can not participate in the restraint of the column and it cannot be considered in the $\Sigma(EI/L)_g$ term of the equation for G. Only FR connections can be used directly in the determination of G. PR connections with a documented moment-rotation response can be utilized, but the $(EI/L)_g$ of each beam must be adjusted to account for the connection flexibility. The ASCE Task Committee on Effective Length (ASCE, 1997) provides a detailed discussion of frame stability with PR connections.

Amplified First-Order Elastic Analysis (Section C2.1b). In this application of the effective length factor, K is used in the determination of the elastic critical buckling load, P_{e1}, for a member, or ΣP_{e2}, for a building story. These elastic critical buckling loads are then used for calculation of the corresponding amplification factors B_1 and B_2.

B_1 is used to estimate the P-δ effects on the nonsway moments, M_{nt}, in axially loaded members. K_1 is calculated in the plane of bending on the basis of no translation of the ends of the member and is normally set to 1.0, unless a smaller value is justified on the basis of analysis. There are also P-δ effects on the sway moments, M_{lt}, as explained previously in the discussion of Equation C2-6b.

B_2 is used to determine the P-Δ effect on the various components of moment, braced and/or combined framing systems. K_2 is calculated in the plane of bending through a sidesway buckling analysis. K_2 may be determined from the sidesway uninhibited alignment chart, Figure C-C2.4, without any correction for story buckling discussed later. ΣP_{e2} from the lateral load resisting columns with K_2 calculated in this way is an accurate estimate of the story elastic sidesway buckling strength. The contribution to the story sidesway buckling strength from leaning columns is zero, and therefore, these columns are not included in the summation in Equation C2-6a. However, the total story vertical load, including all columns in the story, is used for $\alpha\Sigma P_{nt}$ in Equation C2-3.

Since the amplified first-order elastic analysis involves the calculation of elastic buckling loads as a measure of frame and column stiffness, only elastic K factors are appropriate for this use.

Column Flexural Buckling Strength, P_n (Chapter E). In this application of effective length factors, K is used in the determination of the column flexural buckling strength, P_n, which may be based either in an elastic or inelastic buckling analysis.

The column elastic buckling stress, F_e, or the corresponding column axial force at incipient story elastic sidesway buckling, P_e, may be used directly in the calculation of the column flexural buckling strength, P_n. This is because the column strength equations of Chapter E (Equations E3-2 and E3-3) are a function of the ratio F_e/F_y. In fact, if the column axial stress at incipient buckling, F_e, is determined from any appropriate system buckling model, this value of F_e is all that is needed for the calculation of P_n.

The elastic column buckling stress, F_e, is given by Equation E3-4 as shown below:

$$F_e = \frac{\pi^2 E}{\left(\dfrac{KL}{r}\right)^2} \tag{C-C2-3}$$

This equation uses the effective length factor, K, determined by a buckling analysis of a braced frame or a moment frame. F_e can also be obtained directly from a buckling analysis in which the column buckling load is P_e and

$$F_e = \frac{P_e}{A_g} \tag{C-C2-4}$$

Other approaches for the determination of the effective length factor and the critical buckling load using simplified relationships have been presented in the literature. Several of these will be discussed later in this section.

Braced Frames: If $K < 1$ is used for the calculation of P_n in braced frames, the additional demands on stability bracing and the influence on the second-order moments in beams providing restraint to the columns must be considered. This Specification does not address the additional demands on bracing members from the use of $K < 1$. Generally, a rigorous second-order elastic analysis is necessary for calculation of the second-order moments in beams providing restraint to column members designed based on $K < 1$. Therefore, design using $K = 1$ is recommended, except in those special situations where the additional calculations are deemed justified.

Moment Frames: It is important to recognize that sidesway instability of a moment frame is a story phenomenon involving the sum of the sway resistances of each column in the story and the sum of the factored gravity loads in the columns in that story. No individual column in a story can buckle in a sidesway mode without all the columns in that story also buckling. If each column in a story of an moment frame is designed to support its own P and P-Δ moment such that the contribution of each column to the lateral stiffness, or to the story buckling load, is proportional to the axial load supported by the column, then all the columns will buckle simultaneously. Under this condition, there is no interaction among the columns in the story; column sway instability and frame instability occur at the same time. However, many common framing systems can be used that redistribute the story P-Δ effects to the columns in that story in proportion to their individual stiffnesses. This redistribution can be accomplished using such elements as floor diaphragms or horizontal trusses. In a moment frame that contains columns that contribute little or nothing to the sway stiffness of the story, such columns are referred to as leaning columns and they can be designed using $K = 1.0$. The other columns in the story must be designed to support the destabilizing P-Δ moments developed from the loads on these leaning columns. Similarly, the more highly loaded columns in a story will redistribute some of their P-Δ moments to the more lightly loaded columns. This phenomenon must be considered in the determination of K and F_e for all the columns in the story for the design of moment frames. The proper K-factor for calculation of P_n in moment frames, accounting for these effects, is denoted in the following by the symbol K_2.

Two methods for evaluating story frame stability, as measured by ΣP_{e2} for a story, are recognized: the story stiffness method (LeMessurier, 1976; LeMessurier, 1977) and the story buckling method (Yura, 1971). These are reflected in Chapter C with Equations C2-6b and C2-6a, respectively.

For the story stiffness method, K_2 is defined by

$$K_2 = \sqrt{\frac{\Sigma P_r}{(0.85 + 0.15 R_L)\, P_r}\left(\frac{\pi^2 EI}{L^2}\right)\left(\frac{\Delta_H}{\Sigma HL}\right)} \geq \sqrt{\frac{\pi^2 EI}{L^2}\left(\frac{\Delta_H}{1.7 HL}\right)} \qquad \text{(C-C2-5)}$$

This value of K_2 may be used in Equation C-C2-3 or directly in the equations of Chapter E. It is possible that certain columns, having only a small contribution to the lateral load resistance in the overall frame, will have a K_2 value less than 1.0 based on the term to the left of the inequality. The limit on the right-hand side is a minimum value for K_2 that accounts for the interaction between sidesway and nonsidesway buckling (ASCE Task Committee on Effective Length, 1997; White and Hajjar, 1997a). The term H is the shear in the column under consideration, produced by the lateral forces used to compute Δ_H.

It is important to note that this equation for K_2 is not appropriate for use in Equation C2-6a for determining ΣP_{e2} and B_2 in Section C2.1b. It has been derived only for the determination of P_n defined in Chapter E.

Alternatively, Equation C-C2-5 can be reformulated to obtain the column buckling load for use in Equation C-C2-4 as

$$P_{e2} = \left(\frac{\Sigma HL}{\Delta_H}\right)\frac{P_r}{\Sigma P_r}(0.85 + 0.15 R_L) \leq 1.7 HL/\Delta_H \qquad \text{(C-C2-6)}$$

$$R_L = \frac{\Sigma P_r \textit{ leaning columns}}{\Sigma P_r \textit{ all columns}} \qquad \text{(C-C2-7)}$$

ΣP_r in Equations C-C2-5 and C-C2-6 includes all columns in the story, including any leaning columns, and P_r is for the column under consideration. The column load, P_{e2}, calculated from Equation C-C2-6 may be larger than $\pi^2 EI/L^2$ but may not be larger than the limit on the right hand side of this equation. R_L is the ratio of the vertical column load for all leaning columns in the story to the vertical load of all the columns in the story. This factor approaches 1.0 for systems with a large percentage of leaning columns. The purpose of R_L is to account for the debilitating influence of the P-δ effect on the sidesway stiffness of the columns of a story.

Note that ΣP_{e2} given by Equation C2-6b in the story stiffness method is expressed in terms of a building's story drift ratio Δ_H/L from a first-order lateral load analysis at a given applied lateral load level. In preliminary design, this may be taken in terms of a target maximum value for this drift ratio. This approach focuses the engineer's attention on the most fundamental stability requirement in building frames, providing adequate overall story stiffness in relation to the total vertical load, $\alpha \Sigma P_r$, supported by the story. The elastic story stiffness expressed in terms of the drift ratio and the total horizontal load acting on the story is $\Sigma H/(\Delta_H/L)$.

Story Buckling Method. For the story buckling method, K_2 is defined by

$$K_2 = \sqrt{\frac{\pi^2 EI/L^2}{P_r}\left(\frac{\Sigma P_r}{\Sigma \dfrac{\pi^2 EI}{(K_{n2}L)^2}}\right)} \geq \sqrt{\frac{5}{8}}K_{n2} \qquad \text{(C-C2-8)}$$

where K_{n2} is defined as the K value determined directly from the alignment chart in Figure C-C2.4. Again, the value for K_2 calculated from the above equation may be less than 1.0. The limit on the right hand side of this equation is a minimum value for K_2 that accounts for the interaction between sidesway and nonsidesway buckling (ASCE Task Committee on Effective Length, 1997; White and Hajjar, 1997a; Geschwindner, 2002; AISC-SSRC, 2003). It is again important to note that this equation for K_2 is *not* appropriate for use in Equation C2-6a for determining ΣP_{e2} and B_2 in Section C2.1b. It has been derived only for the determination of P_n defined in Chapter E.

Alternatively, Equation C-C2-8 can be reformulated to obtain the column buckling load for use in Equation C-C2-4 as

$$P_{e2} = \left(\frac{P_r}{\Sigma P_r}\right)\Sigma\frac{\pi^2 EI}{(K_{n2}L)^2} \leq 1.6\frac{\pi^2 EI}{(K_{n2}L)^2} \qquad \text{(C-C2-9)}$$

The column load, P_{e2}, calculated from Equation C-C2-9, may be greater than $\pi^2 EI/L^2$ but may not be larger than the limit on the right-hand side of this equation. ΣP_r in Equations C-C2-8 and C-C2-9 includes all columns in the story, including any leaning columns, and P_r is for the column under consideration. K_{n2} in Equations C-C2-8 and C-C2-9 above is determined from the alignment chart in

Figure C-C2.4. Note also that the value of P_n, calculated using K_2 by either method cannot be taken greater than P_n, based on sidesway inhibited buckling. Other methods to calculate K_2 are given in previous editions of this commentary and are discussed elsewhere (ASCE Task Committee on Effective Length, 1997; White and Hajjar, 1997a; Geschwindner, 2002; AISC-SSRC, 2003).

Another simple formula for K_2 (LeMessurier, 1995), based only on the column end moments, is shown below:

$$K_2 = \left[1 + (1 - M_1/M_2)^4\right]\sqrt{1 + \frac{5}{6}\frac{\Sigma P_r\ leaning\ columns}{\Sigma P_r\ nonleaning\ columns}} \quad \text{(C-C2-10)}$$

M_1 is the smaller and M_2 the larger end moment in the column. These moments are determined from a first-order analysis of the frame under wind load. Column inelasticity is considered in the derivation of this equation. The unconservative error in P_n using the above equation is less than 3 percent, as long as the following inequality is satisfied:

$$\left(\frac{\Sigma P_y\ nonleaning\ columns}{\Sigma HL/\Delta_H}\right)\left(\frac{\Sigma P_r\ all\ columns}{\Sigma P_r\ nonleaning\ columns}\right) \le 0.45 \quad \text{(C-C2-11)}$$

As with the other approaches for determining K_2 in this section, this equation for K_2 is not appropriate for use in Equation C2-6a for determining ΣP_{e2} and B_2 in Section C2.1b.

Adjustments in K_2 for Column Inelasticity and Determination of P_n. Adjustments in the effective length factor, K_2, or the column buckling stress, F_e, in the calculation of the column strengths, P_n, can be made based on an inelastic buckling analysis of the frame and the inelasticity inherent in the column under the governing load combination (Yura, 1971; ASCE Task Committee on Effective Length, 1997). Columns loaded into the inelastic range of behavior can be viewed as having a tangent modulus, E_T, that is smaller than E. For such columns, $E_c = E_T$ in the equation for G, which usually gives smaller G values, and therefore, smaller K-factors than those based on elastic behavior. Note that it is usually conservative to base the calculation of P_n on elastic K-factors. For more accurate solutions, inelastic K-factors can be determined from the alignment chart method by using τ_a times E_c for E_c in the equation for G where $\tau_a = E_T/E$ is the column inelastic stiffness reduction factor. Depending on how it is calculated, τ_a may account for both a reduction in the stiffness of columns due to geometric imperfections and spread of plasticity from residual stresses under high compression loading:

(a) For $P_n/P_y \le 0.39$ (elastic):

$$\tau_a = 1.0$$

(b) For $P_n/P_y > 0.39$ (inelastic):

$$\tau_a = -2.724(P_n/P_y)\ln(P_n/P_y) \quad \text{(C-C2-12)}$$

where P_y is the column squash load, F_yA_g, and P_n is the nominal column strength. It should be noted the determination of τ_a is in general an iterative process because

P_n (a function of F_e) is dependent upon τ_a. A conservative simplification that eliminates this iterative process is to use $\alpha P_r/\phi_c$ in place of P_n.

Column inelasticity can be considered in determining K_2 (Equations C-C2-5 and C-C2-8) or P_{e2} (Equations C-C2-6 and C- C2-9) for the story stiffness method and the story buckling method. In the story stiffness method, $\tau_a I_c$ can be substituted for I_c for all columns in the frame analysis used to determine Δ_H. In addition, $\tau_a I_c$ can be used in place of I in Equation C-C2-5. In the story buckling method, τ_a is used in the determination of K_{n2} from the alignment chart in Equations C-C2-8 and C-C2-9 and also in those same equations by replacing I_c with $\tau_a I_c$.

If the column inelastic buckling load (P_{e2} from Equations C-C2-6 and C- C2-9 above, modified for inelasticity as described in the above paragraph) is used to determine F_e from Equation C-C2-4 for use in Chapter E (Equations E3-2 and E3-3), then its value must be divided by τ_a as shown below:

$$F_e = \frac{P_{e2}\ (inelastic)}{\tau_a A_g} \qquad \text{(C-C2-13)}$$

The term in the numerator of the above equation denotes the load in the column at incipient inelastic buckling (ASCE Task Committee on Effective Length, 1997). Alternatively, if an inelastic K_2 is determined using τ_a as described in the previous paragraph, this K factor may be substituted directly into Equation C-C2-3 for calculation of F_e.

Some Conclusions Regarding K. It is important to note that column design using K-factors can be tedious and confusing for complex building structures containing leaning columns and/or combined framing systems, particularly where column inelasticity is considered. This confusion can be avoided if the Direct Analysis Method of Appendix 7 is used, where P_n is always based on $K = 1.0$. Also, the first-order elastic design-analysis method of Section C2.2b is based on the Direct Analysis Method, and hence also uses $K = 1.0$ in the determination of P_n. Furthermore, under certain circumstances where B_2 is sufficiently low, a K-factor of 1.0 may be assumed in design by second-order analysis as specified in Section C2.2a (4). For frames that satisfy this clause, it is not appropriate to use $K = 1.0$ in the calculation of B_2 using Equations C2-6a and C2-3. The use of Equation C2-6b is recommended for the calculation of B_2 within this context.

CHAPTER D

DESIGN OF MEMBERS FOR TENSION

The provisions of Chapter D do not account for eccentricities between the lines of action of connected assemblies.

D1. SLENDERNESS LIMITATIONS

The advisory upper limit on slenderness in the User Note is based on professional judgment and practical considerations of economics, ease of handling and care required so as to minimize inadvertent damage during fabrication, transport, and erection. This slenderness limit is not essential to the structural integrity of tension members; it merely assures a degree of stiffness such that undesirable lateral movement ("slapping" or vibration) will be unlikely. Out-of-straightness within reasonable tolerances does not affect the strength of tension members. Applied tension tends to reduce, whereas compression tends to amplify, out-of-straightness.

For single angles, the radius of gyration about the z-axis produces the maximum l/r and, except for very unusual support conditions, the maximum Kl/r.

D2. TENSILE STRENGTH

Because of *strain hardening*, a ductile steel bar loaded in axial tension can resist without rupture a force greater than the product of its gross area and its specified minimum yield stress. However, excessive elongation of a tension member due to uncontrolled yielding of its gross area not only marks the limit of its usefulness but can precipitate failure of the structural system of which it is a part. On the other hand, depending upon the reduction of area and other mechanical properties of the steel, the member can fail by rupture of the net area at a load smaller than required to yield the gross area. Hence, general yielding of the gross area and rupture of the net area both constitute limit states.

The length of the member in the net area is generally negligible relative to the total length of the member. *Strain hardening* is easily reached in the vicinity of holes and yielding of the net area at fastener holes does not constitute a limit state of practical significance.

Except for HSS that are subjected to *cyclic load* reversals, there is no information that the factors governing the strength of HSS in tension differ from those for other structural shapes, and the provisions in Section D2 apply. Because the number of different end connection types that are practical for HSS is limited, the determination of the net effective area A_e can be simplified using the provisions in Chapter K.

D3. AREA DETERMINATION

1. Gross Area

For HSS, ASTM A500 tolerances allow for a wall thickness that is not greater than $\pm$ 10 percent under thickness; consequently the gross area for ASTM A500 HSS is to be computed using 93 percent of the nominal wall thickness. This reduction is included in the tabulated properties for these sections that are included in the AISC *Manual of Steel Construction* (AISC, 2005a).

2. Net Area

The critical net area is based on net width and load transfer at a particular chain. Because of possible damage around a hole during drilling or punching operations, $^1/_{16}$ in. (1.5 mm) is added to the nominal hole diameter when computing the critical net area.

3. Effective Net Area

Section D3.3 deals with the effect of shear lag, applicable to both welded and bolted tension members. The reduction coefficient U is applied to the net area A_n of bolted members and to the gross area A_g of welded members. As the length of the connection l is increased, the shear lag effect diminishes. This concept is expressed empirically by the equation for U. Using this expression to compute the effective area, the estimated strength of some 1,000 bolted and riveted connection test specimens, with few exceptions, correlated with observed test results within a scatterband of $\pm$10 percent (Munse and Chesson, 1963). Newer research provides further justification for the current provisions (Easterling and Gonzales, 1993).

For any given profile and configuration of connected elements, $\bar{x}$ is the perpendicular distance from the connection plane, or face of the member, to the centroid of the member section resisting the connection force, as shown in Figure C-D3.1. The length l is a function of the number of rows of fasteners or the length of weld. The length l is illustrated as the distance, parallel to the line of force, between the first and last row of fasteners in a line for bolted connections. The number of bolts in a line, for the purpose of the determination of l, is determined by the line with the maximum number of bolts in the connection. For staggered bolts, the out-to-out dimension is used for l, as shown in Figure C-D3.2.

There is insufficient data for establishing a value of U if all lines have only one bolt, but it is probably conservative to use A_e equal to the net area of the connected element. The limit states of block shear (Section J4.3) and bearing (Section J3.10), which must be checked, will probably control the design.

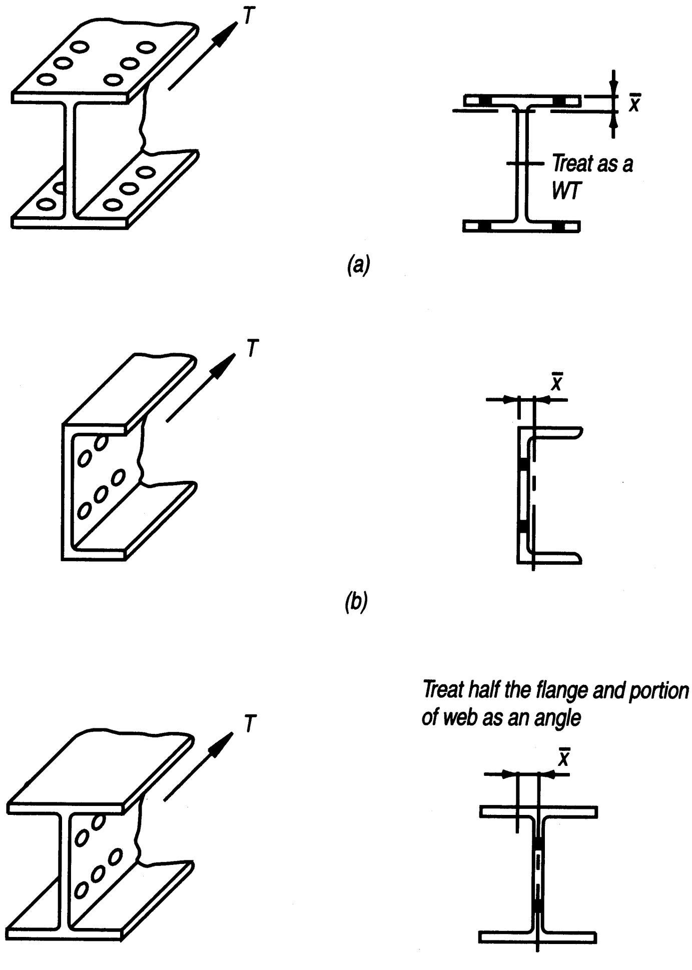


Fig. C-D3.1. Determination of $\bar{x}$ for U.

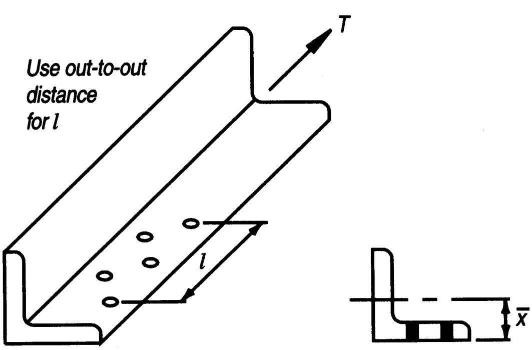


Fig. C-D3.2. Determination of l for U for bolted connections with staggered holes.

Significant eccentricity may exist within the connection if U is less than 0.6. For values of U less than 0.6 the connection may be used only if the provisions for members subject to combined bending and axial force are satisfied in the design of the member.

For welded connections, l is the length of the weld parallel to the line of force as shown in Figure C-D3.3 for longitudinal and longitudinal plus transverse welds.

End connections for HSS in tension are commonly made by welding around the perimeter of the HSS; in this case, there is no shear lag or reduction in the gross area. Alternatively, an end connection with gusset plates can be used. Single gusset plates may be welded in longitudinal slots that are located at the centerline of the cross section. Welding around the end of the gusset plate may be omitted for statically loaded connections to prevent possible *undercutting* of the gusset and having to bridge the gap at the end of the slot. In such cases, the net area at the end of the slot is the critical area as illustrated in Figure C-D3.4. Alternatively, a

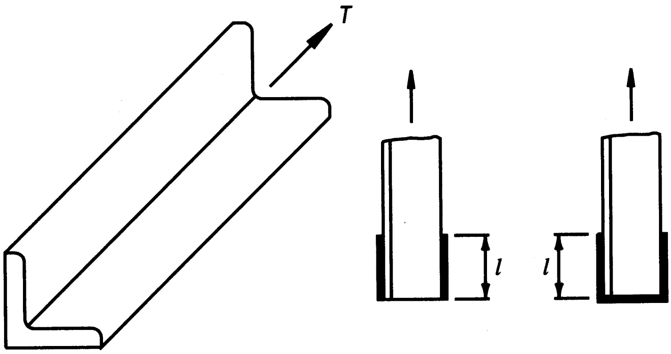


Fig. C-D3.3. Determination of l for U for connections with longitudinal and transverse welds.

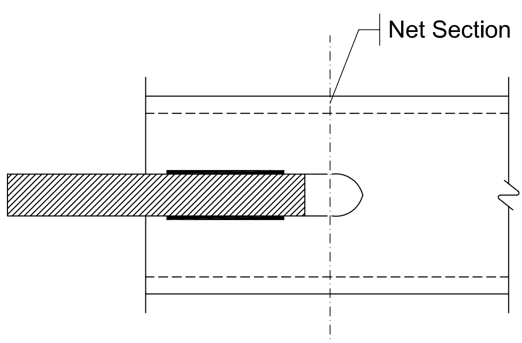


Fig. C-D3.4. Net area through slot for single gusset plate.

pair of gusset plates can be welded to opposite sides of a rectangular HSS with flare bevel groove welds with no reduction in the gross area.

For end connections with gusset plates, the general provisions for shear lag in Case 2 of Table D3.1 can be simplified and the connection eccentricity $\bar{x}$ can be explicitly defined as in Cases 5 and 6. In Cases 5 and 6 it is implied that the weld length, l, should not be less than the depth of the HSS. This is consistent with the weld length requirements in Case 4. In Case 5, the use of $U = 1$ when $l \geq 1.3D$ is based on research (Cheng and Kulak, 2000) that shows that fracture occurs only in short connections and that, in long connections, the round HSS tension member necks within its length and failure is by member yielding and eventual fracture.

The shear lag factors given in Cases 7 and 8 of Table D3.1 were located in the commentary of the 1999 *LRFD Specification* (AISC, 2000b) and are now given as alternate U values to the value determined from $1 - \bar{x}/l$ given for Case 2 in Table D3.1. It is permissible to use the larger of the two values.

D4. BUILT-UP MEMBERS

Although not commonly used, built-up member configurations using lacing, tie plates and perforated cover plates are permitted by this Specification. The length and thickness of tie plates are limited by the distance between the lines of fasteners, h, which may be either bolts or welds.

D5. PIN-CONNECTED MEMBERS

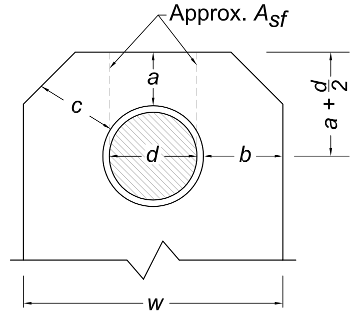
Pin-connected members are occasionally used as tension members with very large dead loads. Pin-connected members are not recommended when there is sufficient variation in live loading to cause wearing of the pins in the holes. The dimensional requirements presented in Specification Section D5.2 must be met to provide for the proper functioning of the pin.

1. Tensile Strength

The tensile strength requirements for pin-connected members use the same ϕ and Ω values as elsewhere in this Specification for similar limit states. However, the definitions of effective net area for tension and shear are different, as shown in Figure C-D5.1.

2. Dimensional Requirements

Dimensional requirements for pin-connected members are illustrated in Figure C-D5.1.



Dimensional Requirements

1. $a \geq 4/3\ b_{eff}$
2. $w \geq 2b_{eff} + d$
3. $c \geq a$

where

$$b_{eff} = 2t + 0.625 \text{ in. (16 mm)} \leq b$$

Fig. C-D5.1. Dimensional requirements for pin-connected members.

D6. EYEBARS

Forged eyebars have generally been replaced by pin-connected plates or eyebars thermally cut from plates. Provisions for the proportioning of eyebars contained in this Specification are based upon standards evolved from long experience with forged eyebars. Through extensive destructive testing, eyebars have been found to provide balanced designs when they are thermally cut instead of forged. The more conservative rules for pin-connected members of nonuniform cross section and for members not having enlarged "circular" heads are likewise based on the results of experimental research (Johnston, 1939).

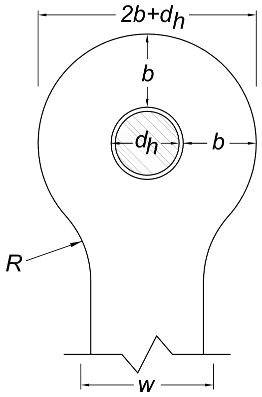
Stockier proportions are required for eyebars fabricated from steel having a yield stress greater than 70 ksi (485 MPa) to eliminate any possibility of their "dishing" under the higher design stress.

1. Tensile Strength

The tensile strength of eyebars is determined as for general tension members, except that, for calculation purposes, the width of the body of the eyebar is limited to eight times its thickness.

2. Dimensional Requirements

Dimensional limitations for eyebars are illustrated in Figure C-D6.1.



Dimensional Requirements

$t \geq 1/2$ in. (13mm) (Exception is provided in D6.2)

$w \leq 8t$

$d \geq 7/8\,w$

$d_h \leq d + 1/32$ in. (1mm)

$R \geq d_h + 2b$

$2/3\,w \leq b \leq 3/4\,w$ (Upper limit is for calculation purposes only)

Fig. C-D6.1. Dimensional limitations for eyebars.

CHAPTER E

DESIGN OF MEMBERS FOR COMPRESSION

E1. GENERAL PROVISIONS

The basic column equations in Section E3 are based on a reasonable conversion of research data into strength equations (Tide, 1985; Tide, 2001). These equations are essentially the same as those in the three previous editions of the *LRFD Specification* (see the discussion in Commentary Section E3 for further discussion). The one significant difference between the previous *LRFD Specifications* and this Specification is that the resistance factor ϕ has been increased from 0.85 to 0.90. The reasons for this increase are the changes in industry practice since the original calibrations were performed in the 1970s.

In the original research on the probability-based strength of steel columns (Bjorhovde, 1972; Bjorhovde, 1978) three *column curves* were recommended. These three *column curve*s were the mean equations of data bands of columns of similar manufacture. For example, hot-formed and cold-formed heat treated HSS columns fell into the data band of highest strength [SSRC Column Category P1 in Galambos (1998), Chapter 3], while welded built-up wide-flange columns made from universal mill plates were included in the data band of lowest strength (SSRC Column Category P3). The largest group of data clustered around SSRC Column Category P2. Had the original *LRFD Specification* opted for using all three *column curves* for the respective column categories, probabilistic analysis would have resulted in a resistance factor equal to $\phi = 0.90$ (Galambos, 1983; Galambos, 1998). It was decided, however, to employ only one *column curve*, SSRC Column Category P2, for all column types. This resulted in a larger data spread and thus in a larger coefficient of variation, and so a resistance factor $\phi = 0.85$ was adopted for the column equations to achieve a reliability comparable to that of beams.

The single *column curve* and the resistance factor of 0.85 were selected by the AISC Committee on Specifications in 1981 when the first draft of the *LRFD Specification* was developed (AISC, 1986). Since then there have been a number of changes in industry practice: (1) welded built-up shapes are no longer manufactured from universal mill plates; and (2) the yield strength of steel has increased with the standard constructional steel (ASTM 992) having a nominal yield stress of 50 ksi (345 MPa). The spread of the yield stress, in other words, its coefficient of variation, has been reduced (Bartlett and others, 2003).

An examination of the SSRC *Column Curve* Selection Table [Figure 3.27 in Galambos (1998)] reveals that there is no longer any SSRC P3 *Column Curve* Category. It is now possible to conservatively use only the statistical data for SSRC Column

Category P2 for the probabilistic determination of the reliability of columns. The curves in Figures C-E1.1 and C-E1.2 show the variation of the reliability index β with the live-to-dead load ratio L/D in the range of 1 to 5 for LRFD with $\phi = 0.90$ and ASD with $\Omega = 1.67$, respectively, for $F_y = 50$ ksi (345 MPa). The reliability index does not fall below $\beta = 2.6$. This is comparable to the reliability of beams. The ASD method gives higher reliability in the lower L/D range than the LRFD method.

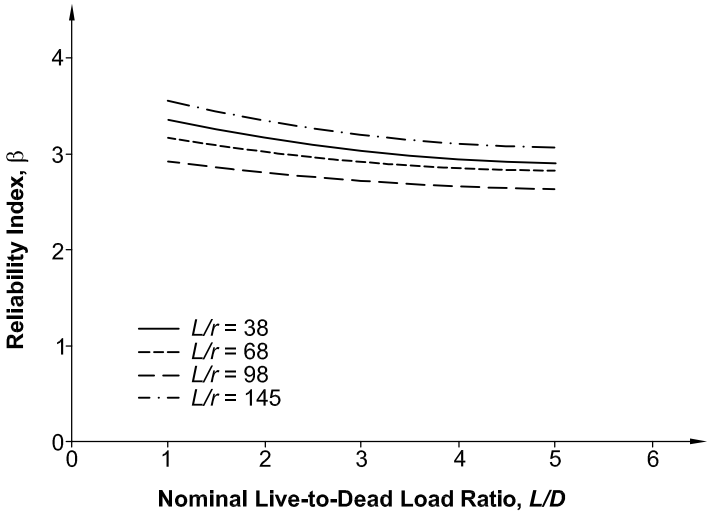


Fig. C-E1.1. Reliability of columns (LRFD).

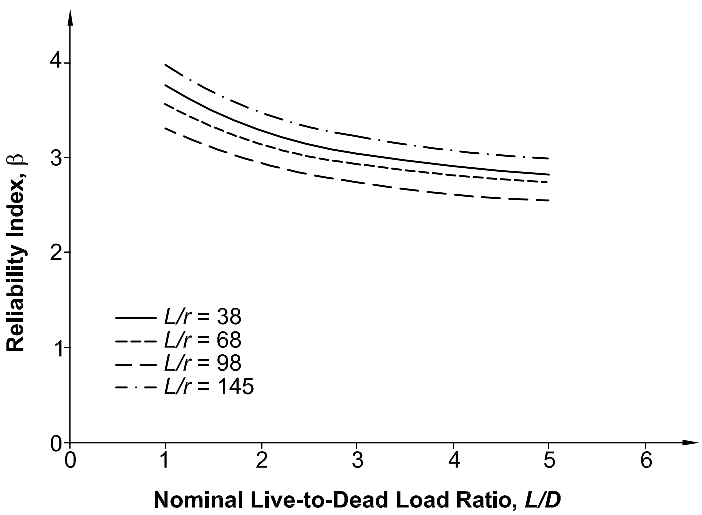


Fig. C-E1.2. Reliability of columns (ASD).

E2. SLENDERNESS LIMITATIONS AND EFFECTIVE LENGTH

The concept of a maximum limiting slenderness ratio has experienced an evolutionary change from a mandatory "... The slenderness ratio, KL/r, of compression members shall not exceed 200 ... " in the 1978 Specification to no restriction at all in this Specification. The 1978 *ASD* and the 1999 *LRFD Specifications* (AISC, 1978; AISC, 2000b) provide a transition from the rigid mandatory limit to no limit by the flexible provision that "... the slenderness ratio, KL/r, preferably should not exceed 200. ..." This latter restriction is actually no limit at all, so the present Specification has disposed with the provision altogether. However, the designer should keep in mind that columns with a slenderness ratio of more than 200 will have a critical stress (Equation E3-4) less than 6.3 ksi (43.5 MPa). The traditional upper limit of 200 was based on professional judgment and practical construction economics, ease of handling, and care required to minimize inadvertent damage during fabrication, transport and erection. It is not recommended to exceed this limit for compression members except for cases where special care is exercised by the fabricator and erector.

E3. COMPRESSIVE STRENGTH FOR FLEXURAL BUCKLING OF MEMBERS WITHOUT SLENDER ELEMENTS

Section E3 applies to compression members with compact and noncompact sections, as defined in Section B4.

The column strength equations in Section E3 are the same as those in the previous editions of the LRFD Specification, with the exception of the cosmetic replacement of the nondimensional slenderness ratio $\lambda_c = \frac{KL}{\pi r}\sqrt{\frac{F_y}{E}}$ by the more familiar $\frac{KL}{r}$. For the convenience of those calculating the elastic buckling stress directly, without first calculating K, the limits on use of Equations E3-2 and E3-3 are also provided in terms of F_e.

Comparisons between the previous column design curves and the new one are shown in Figures C-E3.1 and C-E3.2 for the case of $F_y = 50$ ksi (345 MPa). The curves show the variation of the available column strength with the slenderness ratio for LRFD and ASD, respectively. The LRFD curves reflect the change of the resistance factor ϕ from 0.85 to 0.90, as was explained in Commentary Section E1 above. For both LRFD and ASD, the new column equations give somewhat more economy than the previous editions of the Specification.

The limit between elastic and inelastic buckling is defined to be $\frac{KL}{r} = 4.71\sqrt{\frac{E}{F_y}}$ or $F_e = 0.44F_y$. For convenience, these limits are defined in Table C-E3.1 for the common values of F_y.

One of the key parameters in the column strength equations is the elastic critical stress, F_e. Equation E3-4 presents the familiar Euler form for F_e. However, F_e can be determined by other means also, including a direct frame buckling analysis, as permitted in Chapter C, or from a torsional or *flexural-torsional* buckling analysis addressed in Section E4.

The column strength equations of Section E3 are generic equations that can be used for frame buckling and for torsional or flexural-torsional buckling (Section E4); they can also be entered with a modified slenderness ratio for single-angle members (Section E5); and they can be modified by the Q-factor for columns with slender elements (Section E7).

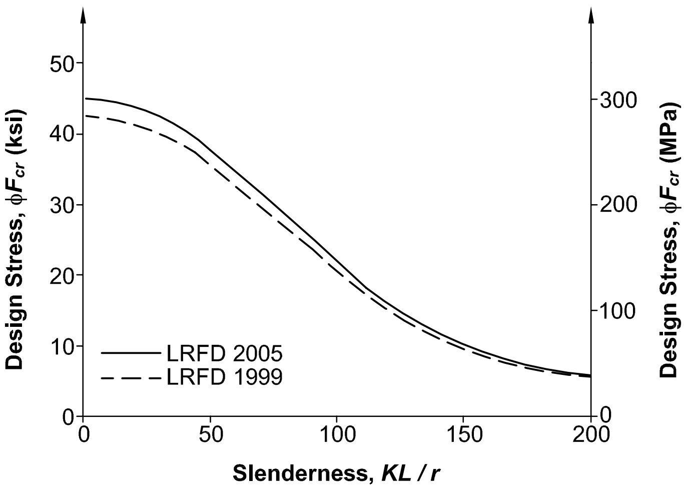


Fig. C-E3.1. LRFD column curves compared.

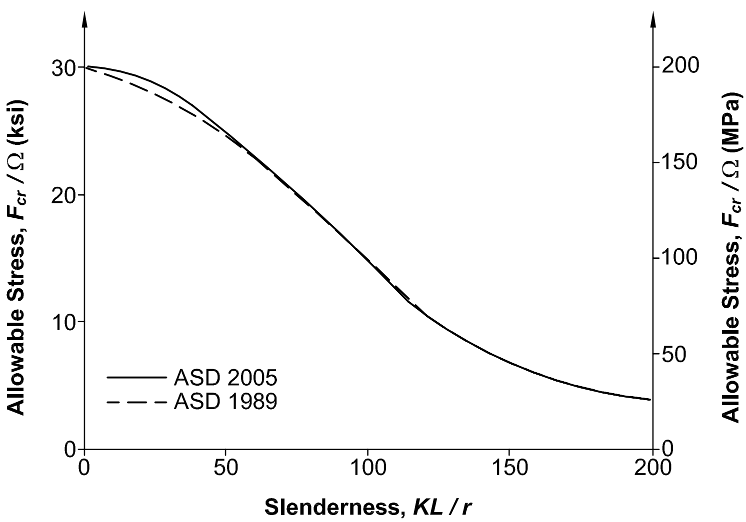


Fig. C-E3.2. ASD column curves compared.

TABLE C-E3.1
Limiting Values of KL/r and F_e

F_y ksi (MPa)	Limiting $\frac{KL}{r}$	F_e ksi (MPa)
36 (248)	134	15.8 (109)
50 (345)	113	22.0 (152)
60 (414)	104	26.4 (182)
70 (483)	96	30.8 (212)

E4. COMPRESSIVE STRENGTH FOR TORSIONAL AND FLEXURAL-TORSIONAL BUCKLING OF MEMBERS WITHOUT SLENDER ELEMENTS

Section E4 applies to singly symmetric and unsymmetric members, and certain doubly symmetric members, such as cruciform or built-up columns, with compact and noncompact sections, as defined in Section B4 for uniformly compressed elements.

The equations in Section E4 for determining the torsional and *flexural-torsional* elastic buckling loads of columns are derived in texts on structural stability [for example, Timoshenko and Gere (1961); Bleich (1952); Galambos (1968); Chen and Atsuta (1977)]. Since these equations apply only to elastic buckling, they must be modified for inelastic buckling by using the torsional and flexural-torsional critical stress, F_{cr}, in the column equations of Section E3.

Torsional buckling of symmetric shapes and flexural-torsional buckling of unsymmetrical shapes are failure modes usually not considered in the design of hot-rolled columns. They generally do not govern, or the *critical load* differs very little from the weak-axis planar buckling load. Torsional and flexural-torsional buckling modes may, however, control the strength of symmetric columns manufactured from relatively thin plate elements and unsymmetric columns and symmetric columns having torsional unbraced lengths significantly larger than the weak-axis flexural unbraced lengths. Equations for determining the critical stress for such columns are given in Section E4. Table C-E4.1 serves as a guide for selecting the appropriate equations.

The simpler method of calculating the buckling strength of double-angle and T-shaped members (Equation E4-2) uses directly the y-axis flexural strength from the column equations of Section E3 (Galambos, 1991). Tees that conform to the limits of Table C-E4.2 need not be checked for flexural-torsional buckling.

Equations E4-4 and E4-11 contain a torsional buckling effective length factor K_z. This factor may be conservatively taken as $K_z = 1.0$. For greater accuracy, $K_z = 0.5$ if both ends of the column have a connection that restrains warping, say by boxing the end over a length at least equal to the depth of the member. If one end

TABLE C-E4.1
Selection of Equations for Torsional and Flexural-Torsional Buckling

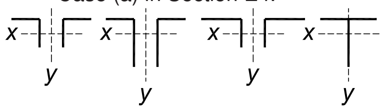
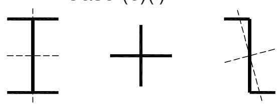
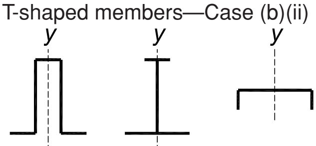
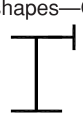
Type of Cross Section	Applicable Equations in Section E4
Double angle and T-shaped members—Case (a) in Section E4. 	E4-2 and E4-3
All doubly symmetric shapes and Z-shapes—Case (b)(i) 	E4-4
Singly symmetric members except double angles and T-shaped members—Case (b)(ii) 	E4-5
Unsymmetrical shapes—Case (b)(iii) 	E4-6

TABLE C-E4.2
Limiting Proportions for Tees

Shape	Ratio of Full Flange Width to Profile Depth	Ratio of Flange Thickness to Stem Thickness
Built-up tees	≥ 0.50	≥ 1.25
Rolled tees	≥ 0.50	≥ 1.10

of the member is restrained from warping and the other end is free to warp, then $K_z = 0.7$.

At points of bracing both lateral and/or torsional bracing shall be provided, as required in Appendix 6. Seaburg and Carter (1997) provides an overview of the fundamentals of torsional loading for structural steel members. Design examples are also included.

E5. SINGLE-ANGLE COMPRESSION MEMBERS

Section E5 addresses the design of single angles subjected to an axial compressive load effect introduced through one connected leg. The attached leg is to be fixed to a gusset plate or the projecting leg of another member by welding or by a bolted connection with at least two bolts. The equivalent slenderness expressions in this section presume significant restraint about the y-axis, which is perpendicular to the connected leg. This leads to the angle member tending to bend and buckle primarily about the x-axis. For this reason L/r_x is the slenderness parameter used. The modified slenderness ratios indirectly account for bending in the angles due to the eccentricity of loading and for the effects of end restraint from the truss chords. The values for box trusses reflect greater rotational end restraint as compared to that provided by planar trusses.

The equivalent slenderness expressions also presume a degree of rotational restraint. Equations E5-3 and E5-4 [Case (b)] assume a higher degree of x-axis rotational restraint than do Equations E5-1 and E5-2 [Case (a)]. Equations E5-3 and E5-4 are essentially equivalent to those employed for equal-leg angles as web members in latticed transmission towers in ASCE 10-97 (ASCE, 2000).

In space trusses, the web members framing in from one face typically restrain the twist of the chord at the panel points and thus provide significant x-axis restraint of the angles under consideration. It is possible that the chords of a planar truss well restrained against twist justify use of Case (b), in other words, Equations E5-3 and E5-4. Similarly, simple single-angle diagonal braces in braced frames could be considered to have enough end restraint such that Case (a), in other words, Equations E5-1 and E5-2 could be employed for their design. This procedure, however, is not intended for the evaluation of the compressive strength of x-braced single angles.

The procedure in Section E5 permits use of unequal-leg angles attached by the smaller leg provided that the equivalent slenderness is increased by an amount that is a function of the ratio of the longer to the shorter leg lengths, and has an upper limit on L/r_z.

If the single-angle compression members cannot be evaluated using the procedures in this section, use the provisions of Section H2. In evaluating P_n, the effective length due to end restraint should be considered. With values of effective length factors about the geometric axes, one can use the procedure in Lutz (1992) to compute an effective radius of gyration for the column. To obtain results that are not too conservative, one must also consider that end restraint reduces the eccentricity of the axial load of single-angle struts and thus the value of f_b used in the flexural term(s) in Equation H2-1.

E6. BUILT-UP MEMBERS

Section E6 addresses the strength and dimensional requirements of built-up members composed of two or more shapes interconnected by stitch bolts or welds.

1. Compressive Strength

The longitudinal spacing of connectors connecting components of built-up compression members must be such that the slenderness ratio L/r of individual shapes does not exceed three-fourths of the slenderness ratio of the entire member. However, this requirement does not necessarily ensure that the effective slenderness ratio of the built-up member is equal to that of a built-up member acting as a single unit. Section E6.1 gives equations for modified slenderness ratios that are based on research and take into account the effect of shear deformation in the connectors (Zandonini, 1985). Equation E6-1 for snug-tight intermediate connectors is empirically based on test results. Equation E6-2 is derived from theory and verified by test data. In both cases the end connection must be welded or fully tensioned bolted (Aslani and Goel, 1991). The connectors must be designed to resist the shear forces that develop in the buckled member. The shear stresses are highest where the slope of the buckled member is the steepest (Bleich, 1952). Fastener spacing less than the maximum required for strength may be needed to ensure a close fit over the entire faying surface of components in continuous contact. Special requirements for weathering steel members exposed to atmospheric corrosion are given in Brockenbrough (1983).

2. Dimensional Requirements

Section E6.2 provides requirements for dimensioning built-up members that cannot be stated in terms of calculated stress but are based upon judgment and experience.

E7. MEMBERS WITH SLENDER ELEMENTS

The structural engineer designing with hot-rolled plates and shapes will seldom find an occasion to turn to Section E7 of the Specification. Among rolled shapes the most frequently encountered cases requiring the application of this section are columns containing angles with thin legs and tee-shaped columns having slender stems. Special attention to the determination of Q must be given when columns are made by welding or bolting thin plates together.

The provisions of Section E7 address the modifications to be made when one or more plate elements in the column cross sections are slender. A plate element is considered to be slender if its width-thickness ratio exceeds the limiting value λ_r defined in Table B4.1. As long as the plate element is not slender, it can support the full yield stress without local buckling. When the cross section contains slender elements, the slenderness reduction factor Q defines the ratio of the stress at local buckling to the yield stress, F_y. The yield stress, F_y, is replaced by the value QF_y in the column equations of Section E3. These equations are repeated as Equations E7-2 and E7-3. This approach to dealing with columns with slender elements has been used since the 1969 Specification (AISC, 1969), emulating the 1969 AISI Specification (AISI, 1969). Prior to 1969, the AISC practice was to remove the width of the plate that exceeded the limit λ_r and check the remaining cross section for conformance with the allowable stress, which proved inefficient and uneconomical. The equations in Section E7 are almost identical to the original equations, with one notable exception that will be discussed subsequently.

This Specification makes a distinction between columns having unstiffened and stiffened elements. Two separate philosophies are used: Unstiffened elements are considered to have attained their limit state when they reach the theoretical local buckling stress. Stiffened elements, on the other hand, make use of the post-buckling strength inherent in a plate that is supported on both of its longitudinal edges, such as in HSS columns. The effective width concept is used to obtain the added post-buckling strength. This dual philosophy reflects 1969 practice in the design of cold-formed columns. Subsequent editions of the AISI Specifications, in particular, the *North American Specification for the Design of Cold-Formed Steel Structural Members* (AISI, 2001), hereafter referred to as the AISI *North American Specification* adopted the effective width concept for both stiffened and unstiffened columns. Following editions of the AISC Specification (including this Specification) did not follow the example set by AISI for unstiffened plates because the advantages of the post-buckling strength do not become available unless the plate elements are very slender. Such dimensions are common for cold-formed columns, but are rarely encountered in structures made from hot-rolled plates.

1. Slender Unstiffened Elements, Q_s

Equations for the slender element reduction factor, Q_s, are given in Section E7.1 for outstanding elements in rolled shapes (Case a), built-up shapes (Case b), single angles (Case c), and stems of tees (Case d). The underlying scheme for these provisions is illustrated in Figure C-E7.1. The curves show the relationship between the Q-factor and a non-dimensional slenderness ratio $\frac{b}{t}\sqrt{\frac{F_y}{E}\frac{12(1-\nu^2)}{\pi^2 k}}$.

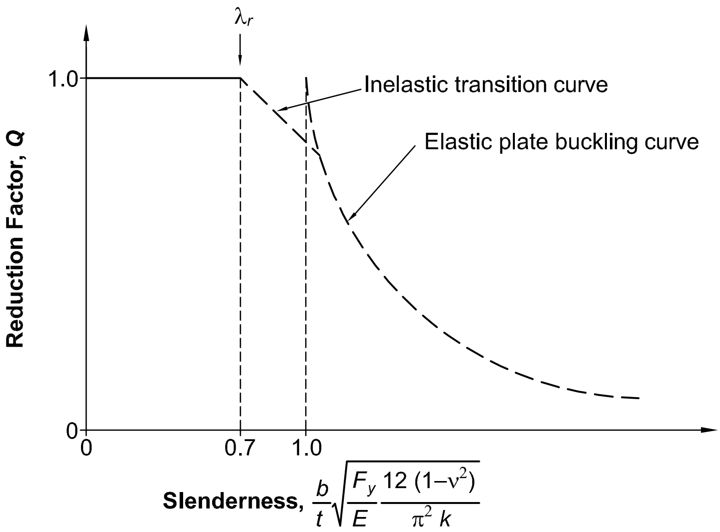


Fig. C-E7.1. Definition of Q_s for unstiffened slender elements.

The width b and thickness t are defined for the applicable cross sections in Section B4; $\nu = 0.3$ (Poisson's ratio), and k is the plate buckling coefficient characteristic of the type of plate edge-restraint. For single angles, $k = 0.425$ (no restraint is assumed from the other leg), and for outstanding flange elements and stems of tees, k equals approximately 0.7, reflecting an estimated restraint from the part of the cross section to which the plate is attached on one of its edges, the other edge being free.

The curve relating Q to the plate slenderness ratio has three components: (i) a part where $Q = 1$ when the slenderness factor is less than or equal to 0.7 (the plate can be stressed up to its yield stress), (ii) the elastic plate buckling portion when buckling is governed by $F_{cr} = \dfrac{\pi^2 Ek}{12(1-\nu^2)\left(\frac{b}{t}\right)^2}$, and (iii) a transition range that empirically accounts for the effect of early yielding due to *residual stresses* in the shape. Generally this transition range is taken as a straight line. The development of the provisions for unstiffened elements is due to the research of Winter and his co-workers, and a full listing of references is provided in the Commentary to the AISI *North American Specification* (AISI, 2001). The slenderness provisions are illustrated for the example of slender flanges of rolled shapes in Figure C-E7.2.

The equations for the unstiffened projecting flanges, angles and plates in built-up cross sections (Equations E7-7 through E7-9) have a history that starts with the research reported in Johnson (1985). It was noted in tests of beams with slender flanges and slender webs that there was an interaction between the buckling of the flanges and the distortions in the web causing an unconservative prediction of strength. A modification based on the equations recommended in Johnson (1985) appeared first in the 1989 *ASD Specification* (AISC, 1989).

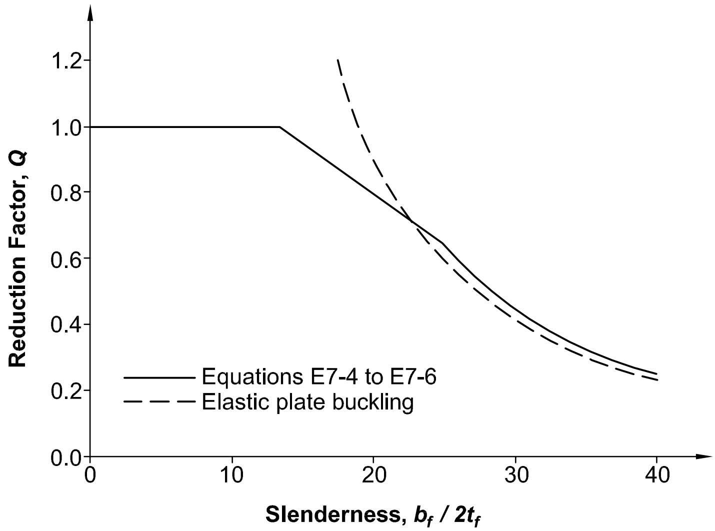


Fig. C-E7.2. Q *for rolled wide-flange columns of* F_y *= 50 ksi (345 MPa).*

Modifications to simplify the original equations were introduced in the 1993 *LRFD Specification* (AISC, 1993), and these equations have remained unchanged in the present Specification. The influence of web slenderness is accounted for by the introduction of the factor

$$k_c = \frac{4}{\sqrt{\dfrac{h}{t_w}}} \tag{C-E7-1}$$

into the equations for λ_r and Q, where k_c shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes.

2. Slender Stiffened Elements, Q_a

While for slender unstiffened elements the Specification for local buckling is based on the limit state of the onset of plate buckling, an improved approach based on the effective width concept is used for the compressive strength of stiffened elements in columns. This method was first proposed in von Kármán, Sechler, and Donnell (1932). This was later modified in Winter (1947) to provide a transition between very slender elements and stockier elements shown by tests to be fully effective. As modified in Winter (1947) for the AISI *North American Specification* (AISI, 2001), the ratio of effective width to actual width increases as the level of compressive stress applied to a stiffened element in a member is decreased, and takes the form

$$\frac{b_e}{t} = 1.9\sqrt{\frac{E}{f}}\left[1 - \frac{C}{(b/t)}\sqrt{\frac{E}{f}}\right] \tag{C-E7-2}$$

where f is taken as F_{cr} of the column based on $Q = 1.0$, and C is a constant based on test results (Winter, 1947).

The basis for cold-formed steel columns in the AISI *North American Specification* editions since the 1970s is $C = 0.415$. The original AISI coefficient 1.9 in Equation C-E7-2 is changed to 1.92 in the Specification to reflect the fact that the modulus of elasticity E is taken as 29,500 ksi (203 400 MPa) for cold-formed steel, and 29,000 ksi (200 000 MPa) for hot-rolled steel.

For the case of square and rectangular box-sections of uniform thickness, where the sides provide negligible rotational restraint to one another, the value of $C = 0.38$ in Equation E7-18 is higher than the value of $C = 0.34$ in Equation E7-17. Equation E7-17 applies to the general case of stiffened plates in uniform compression where there is substantial restraint from the adjacent flange or web elements. The coefficients $C = 0.38$ and $C = 0.34$ are smaller than the corresponding value of $C = 0.415$ in the AISI *North American Specification* (AISI, 2001), reflecting the fact that hot-rolled steel sections have stiffer connections between plates due to welding or fillets in rolled shapes than do cold-formed shapes.

The classical theory of longitudinally compressed cylinders overestimates the actual buckling strength, often by 200 percent or more. Inevitable imperfections of shape and the eccentricity of the load are responsible for the reduction in actual strength below the theoretical strength. The limits in Section E7.2(c) are based upon test evidence (Sherman, 1976), rather than theoretical calculations, that local buckling will not occur if

$$\frac{D}{t} \leq \frac{0.11E}{F_y}$$

When D/t exceeds this value but is less than

$$\frac{D}{t} \leq \frac{0.45E}{F_y}$$

Equation E7-19 provides a reduction in the local buckling reduction factor Q. This Specification does not recommend the use of round HSS or pipe columns with

$$\frac{D}{t} > \frac{0.45E}{F_y}$$

CHAPTER F

DESIGN OF MEMBERS FOR FLEXURE

F1. GENERAL PROVISIONS

Chapter F applies to members subject to simple bending about one principal axis of the cross section. Section F2 gives the provisions for the flexural strength of doubly symmetric compact I-shaped and channel members subject to bending about their major axis. For most designers, the provisions in this section will be sufficient to perform their everyday designs. The remaining sections of Chapter F address less frequently occurring cases encountered by structural engineers. Since there are many such cases, many equations and many pages in the Specification, the table in User Note F1.1 is provided as a map for navigating through the cases considered in Chapter F. The coverage of the chapter is extensive and there are many equations that appear formidable; however, it is stressed again that for most designs, the engineer need seldom go beyond Section F2.

For all sections covered in Chapter F, the highest possible nominal flexural strength is the plastic moment, $M_n = M_p$. Being able to use this value in design represents the optimum use of the steel. In order to attain M_p the beam cross section must be compact and the member must be laterally braced. Compactness depends on the flange and web plate width-to-thickness ratios, as defined in Section B4. When these conditions are not met, the available nominal flexural strength diminishes. All sections in Chapter F treat this reduction in the same way. For laterally braced beams, the plastic moment region extends over the range of plate width-thickness ratios λ terminating at λ_p. This is the compact condition. Beyond these limits the nominal moment reduces linearly until λ reaches λ_r. This is the range where the section is noncompact. Beyond λ_r the section is a slender-element section.

These three ranges are illustrated in Figure C-F1.1 for the case of rolled wide-flange members for the limit state of flange local buckling. The curve in Figure C-F1.1 shows the relationship between the flange width-thickness ratio $b_f/2t_f$ and the nominal flexural strength, M_n.

The basic relationship between the nominal flexural strength, M_n, and the unbraced length, L_b, for the limit state of lateral-torsional buckling is shown in Figure C-F1.2 for a compact section [W27×84 (W690×125), $F_y = 50$ ksi (345 MPa)] subjected to uniform bending, $C_b = 1.0$.

There are four principal zones defined on the basic curve by the lengths L_{pd}, L_p, and L_r. Equation F2-5 defines the maximum unbraced length L_p to reach M_p

with uniform moment. Elastic lateral-torsional buckling will occur when the unbraced length is greater than L_r given by Equation F2-6. Equation F2-2 defines the range of inelastic lateral-torsional buckling as a straight line between the defined limits M_p at L_p and $0.7F_yS_x$ at L_r. Buckling strength in the elastic region is given by Equations F2-3 and F2-4 for I-shaped members. The length

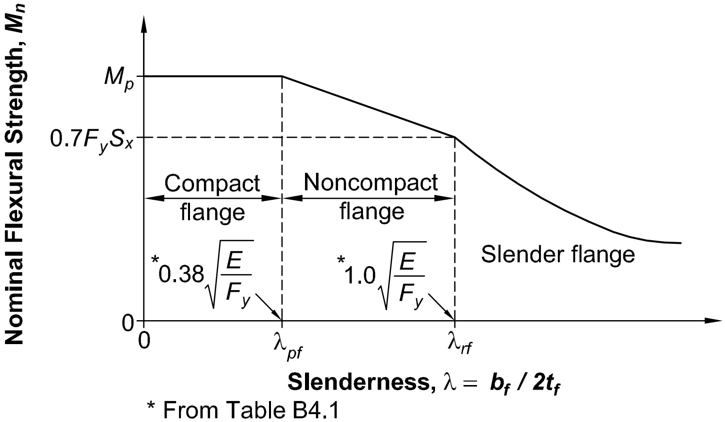


Fig. C-F1.1. Nominal flexural strength as a function of the flange width-thickness ratio of rolled I-shapes.

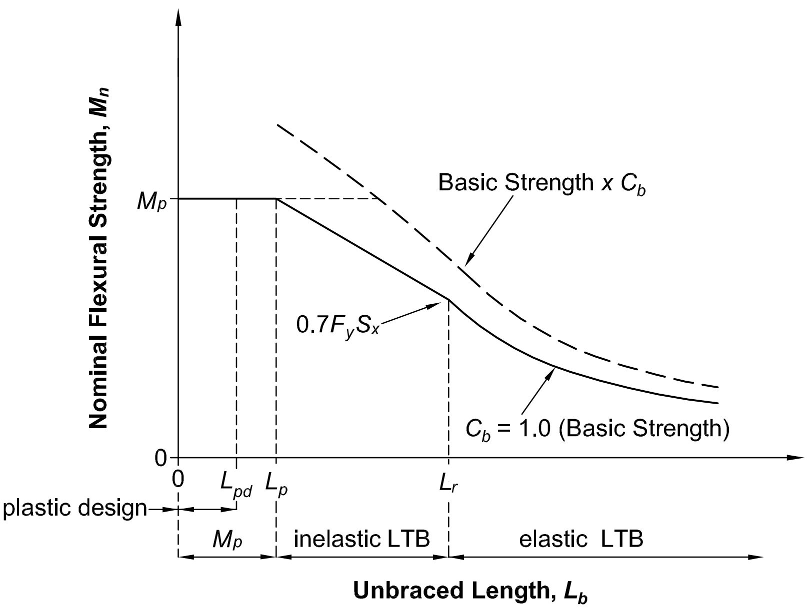


Fig. C-F1.2. Nominal flexural strength as a function of unbraced length and moment gradient.

L_{pd} is defined in Appendix 1 as the limiting unbraced length needed for plastic design.

For moment diagrams along the member other than uniform moment, the lateral buckling strength is obtained by multiplying the basic strength in the elastic and inelastic region by C_b as shown in Figure C-F1.2. However, in no case can the maximum moment capacity exceed the plastic moment M_p. Note that L_p given by Equation F2-5 is merely a definition that has physical meaning only when $C_b = 1.0$. For C_b greater than 1.0, members with larger unbraced lengths can reach M_p, as shown by the curve for $C_b > 1.0$ in Figure C-F1.2. This length is calculated by setting Equation F2-2 equal to M_p and solving for L_b using the actual value of C_b.

The equation

$$C_b = 1.75 + 1.05\left(\frac{M_1}{M_2}\right) + 0.3\left(\frac{M_1}{M_2}\right)^2 \tag{C-F1-1}$$

has been used since 1961 in AISC Specifications to adjust the lateral-torsional buckling equations for variations in the moment diagram within the unbraced length. However, this equation is only applicable to moment diagrams that consist of straight lines between braced points—a condition that is rare in beam design. The equation provides a lower bound to the solutions developed in Salvadori (1956). Equation C-F1-1 can be easily misinterpreted and misapplied to moment diagrams that are not linear within the unbraced segment. Kirby and Nethercot (1979) present an equation that applies to various shapes of moment diagrams within the unbraced segment. Their original equation has been slightly adjusted to give Equation C-F1-2 (Equation F1-1 in the body of the Specification):

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} \tag{C-F1-2}$$

This equation gives a more accurate solution for a fixed-end beam, and gives approximately the same answers as Equation C-F1-1 for moment diagrams with straight lines between points of bracing. C_b computed by Equation C-F1-2 for moment diagrams with other shapes show good comparison with the more precise but also more complex equations (Galambos, 1998). The absolute values of the three quarter-point moments and the maximum moment regardless of its location are used in Equation C-F1-2. The maximum moment in the unbraced segment is always used for comparison with the nominal moment M_n. The length between braces, not the distance to inflection points is used. It is still satisfactory to use C_b from Equation C-F1-1 for straight-line moment diagrams within the unbraced length.

The equations for the limit state of lateral-torsional buckling in Chapter F assume that the loads are applied along the beam centroidal axis. C_b may be conservatively taken equal to 1.0, with the exception of some cases involving unbraced cantilevers or members with no bracing within the span and with significant loading applied to the top flange. If the load is placed on the top flange and the flange is not braced, there is a tipping effect that reduces the critical moment; conversely, if the load is suspended from an unbraced bottom flange, there is a stabilizing effect that increases the critical moment (Galambos, 1998). For unbraced top flange loading on compact I-shaped members, the reduced critical moment may be conservatively approximated by setting the square root expression in Equation F2-4 equal to unity.

An effective length factor of unity is implied in the critical moment equations to represent the worst-case simply supported unbraced segment. Consideration of any end restraint due to adjacent nonbuckled segments on the critical segment can increase its strength. The effects of beam continuity on lateral-torsional buckling have been studied, and a simple conservative design method, based on the analogy to end-restrained nonsway columns with an effective length less than unity, has been proposed (Galambos, 1998).

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

Section F2 applies to members with compact I-shaped or channel cross sections subject to bending about their major axis; hence the only limit state to consider is lateral-torsional buckling. Almost all rolled wide-flange shapes listed in the AISC *Manual of Steel Construction* are eligible to be designed by the provisions of this section, as indicated in the User Note in the Specification.

The equations in Section F2 are identical to the corresponding equations in Section F1 of the 1999 *LRFD Specification* (AISC, 2000b), although they are presented in different form. The following table gives the list of equivalent equations:

TABLE C-F2.1
Comparison of Equations for Nominal Flexural Strength

1999 AISC/LRFD Specification Equations	Current Specification Equations
F1-1	F2-1
F1-2	F2-2
F1-13	F2-3 and F2-4

The only difference between the two specifications is that the stress at the interface between inelastic and elastic buckling has been changed from $F_y - F_r$ in the 1999 edition to $0.7F_y$ herein. In the previous Specification the residual stress, F_r, for rolled and welded shapes was different, namely 10 ksi (69 MPa) and 16.5 ksi (114 MPa), respectively, while in this Specification the *residual stress* was taken as $0.3F_y$ so that the value of $F_y - F_r = 0.7F_y$ was adopted. This change was made in the interest of simplicity with negligible effect on economy.

The elastic lateral-torsional buckling stress, F_{cr}, of Equation F2-4:

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

is identical to Equation F1-13 in the 1999 *LRFD Specification* (AISC, 2000b):

$$F_{cr} = \frac{M_{cr}}{S_x} = \frac{C_b \pi}{L_b S_x} \sqrt{E I_y G J + \left(\frac{\pi E}{L_b}\right)^2 I_y C_w} \qquad \text{(C-F2-2)}$$

if $c = 1$ (see Section F2 for definition) and

$$r_{ts}{}^2 = \frac{\sqrt{I_y C_w}}{S_x}; \quad h_o = d - t_f; \quad \text{and } \frac{2G}{\pi^2 E} = 0.0779$$

Equation F2-5 is the same as F1-4 in the 1999 *LRFD Specification* (AISC, 2000b), and F2-6 corresponds to F1-6. It is obtained by setting $F_{cr} = 0.7F_y$ in Equation F2-4 and solving for L_b. The term r_{ts} can conservatively be calculated as the radius of gyration of the compression flange plus one-sixth of the web.

These provisions have been simplified when compared to the previous ASD provisions based on a more informed understanding of beam limit states behavior. The maximum allowable stress obtained in these provisions may be slightly higher than the previous limit of $0.66F_y$, since the true plastic strength of the member is reflected by use of the plastic section modulus in Equation F2-1. The Section F2 provisions for unbraced length are satisfied through the use of two equations, one for inelastic lateral-torsional buckling (Equation F2-2), and one for elastic lateral-torsional buckling (Equation F2-3). Previous ASD provisions placed an arbitrary stress limit of $0.6F_y$ when a beam was not fully braced and required that three equations be checked with the selection of the largest stress to determine the strength of a laterally unbraced beam. With the current provisions, once the unbraced length is determined, the member strength can be obtained directly from these equations.

F3. DOUBLY SYMMETRIC I-SHAPED MEMBERS WITH COMPACT WEBS AND NONCOMPACT OR SLENDER FLANGES BENT ABOUT THEIR MAJOR AXIS

Section F3 is a supplement to Section F2 for the case where the flange of the section is noncompact or slender (see Figure C-F1.1, linear variation of M_n

between λ_{pf} and λ_{rf}). As pointed out in the user note of Section F2, very few rolled wide-flange shapes are subject to this criterion.

F4. OTHER I-SHAPED MEMBERS WITH COMPACT OR NONCOMPACT WEBS BENT ABOUT THEIR MAJOR AXIS

Section F4 has no direct counterpart in previous AISC Specifications except for the lateral buckling provisions for singly symmetric sections in Table A-F1.1 in the 1999 *LRFD Specification* (AISC, 2000b). These provisions are not carried over to the present Specification. The provisions of Section F4 are applicable to doubly symmetric wide-flange beams with slender flanges and to singly symmetric wide-flange members with compact, noncompact, and slender flanges, and noncompact webs (see the Table in User Note F1.1). This part of Chapter F essentially deals with welded I-shaped beams where the webs are not slender. The following section, F5, considers welded I-shapes with slender webs. The contents of Section F4 are based on White (2004).

Three limit states are considered: (a) lateral-torsional buckling (LTB); (b) flange local buckling (FLB); and (c) tension flange yielding (TFY). The effect of inelastic buckling of the web is taken care of indirectly by multiplying the moment causing yielding in the compression flange by a factor R_{pc} and the moment causing yielding in the tension flange by a factor R_{pt}. These two factors can vary from unity to as high as 1.6. Conservatively, they can be assumed to equal 1.0. The following steps are provided as a guide to the determination of R_{pc} and R_{pt}.

Step 1. Calculate h_p and h_c: See Figure C-F4.1.

Step 2. Determine web slenderness and yield moments in compression and tension:

$$\left\{\begin{array}{l} \lambda = \dfrac{h_c}{t_w} \\ S_{xc} = \dfrac{I_x}{y}; \quad S_{xt} = \dfrac{I_x}{d-y} \\ M_{yc} = F_y S_{xc}; \quad M_{yt} = F_y S_{xt} \end{array}\right\} \tag{C-F4-1}$$

Step 3. Determine λ_{pw} and λ_{rw}

$$\left\{\begin{array}{l} \lambda_{pw} = \dfrac{\dfrac{h_c}{h_p}\sqrt{\dfrac{E}{F_y}}}{\left[\dfrac{0.54M_p}{M_y} - 0.09\right]^2} \le 3.76\sqrt{\dfrac{E}{F_y}} \\ \lambda_{rw} = 5.70\sqrt{\dfrac{E}{F_y}} \end{array}\right\} \tag{C-F4-2}$$

If $\lambda > \lambda_{rw}$ then the web is slender and the design is governed by Section F5.

Step 4. Calculate R_{pc} and R_{pt} by Equations F4-9a or F4-9b and F4-15a or F4-15b, respectively.

The basic maximum nominal moment is $R_{pc}M_{yc} = R_{pc}F_yS_{xc}$ if the flange is in compression, and $R_{pt}M_{yt} = R_{pt}F_yS_{xt}$ if it is in tension. Thereafter, the provisions are the same as for doubly symmetric members in Sections F2 and F3. For the limit state of lateral-torsional buckling, I-shaped members with cross sections that have unequal flanges are treated as if they were doubly symmetric I-shapes. That is, Equations F2-4 and F2-6 are the same as Equations F4-5 and F4-8, except the former use S_x and the latter use S_{xc}, the elastic section moduli of the entire section and of the compression side, respectively. This is a simplification that tends to be somewhat conservative if the compression flange is smaller than the tension flange, and it is somewhat unconservative when the reverse is true. It is also required to check for tension flange yielding if the tension flange is smaller than the compression flange (Section F4.3).

For a more accurate solution, especially when the loads are not applied at the centroid of the member, the designer is directed to Chapter 5 of the SSRC Guide (Galambos, 1998; Galambos, 2001; White and Jung, 2003). White gives the following alternative equations in lieu of Equations F4-5 and F4-8:

$$M_n = C_b \frac{\pi^2 EI_y}{L_b^2}\left\{\frac{\beta_x}{2} + \sqrt{\left(\frac{\beta_x}{2}\right)^2 + \frac{C_w}{I_y}\left[1 + 0.0390\frac{J}{C_w}L_b^2\right]}\right\} \quad \text{(C-F4-3)}$$

$$L_r = \frac{1.38E\sqrt{I_yJ}}{S_{xc}F_{yr}}\sqrt{\frac{2.6\beta_xF_{yr}S_{xc}}{EJ} + 1 + \sqrt{\left[\frac{2.6\beta_xF_{yr}S_{xc}}{EJ}+1\right]^2 + \frac{27.0C_w}{I_y}\left(\frac{F_{yr}S_{xc}}{EJ}\right)^2}} \quad \text{(C-F4-4)}$$

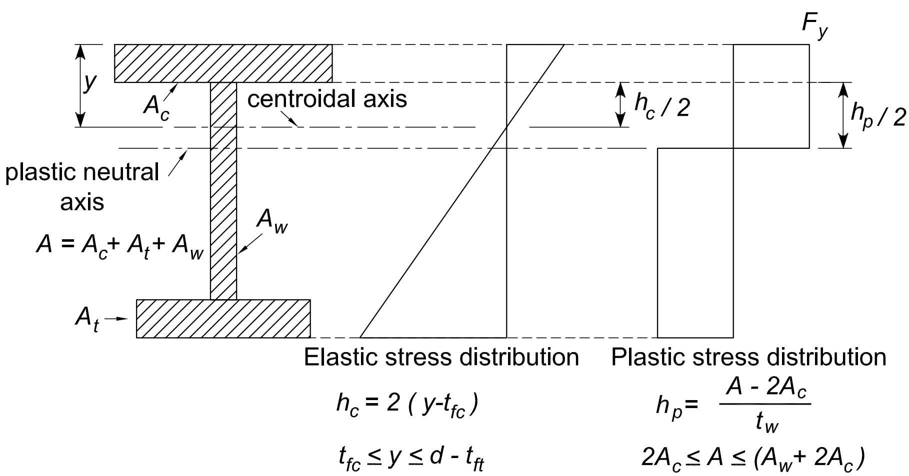


Fig. C-F4.1. Elastic and plastic stress distributions.

where the coefficient of monosymmetry, $\beta_x = 0.9h\alpha\left(\frac{I_{yc}}{I_{yt}} - 1\right)$,

the warping constant, $C_w = h^2 I_{yc}\alpha$, and $\alpha = \dfrac{1}{\dfrac{I_{yc}}{I_{yt}} + 1}$.

F5. DOUBLY SYMMETRIC AND SINGLY SYMMETRIC I-SHAPED MEMBERS WITH SLENDER WEBS BENT ABOUT THEIR MAJOR AXIS

This section applies for doubly and singly symmetric I-shaped welded plate girders with a slender web, that is, $\frac{h_c}{t_w} > \lambda_r = 5.70\sqrt{\frac{E}{F_y}}$. The applicable limit states are lateral-torsional buckling, compression flange local buckling and tension flange local yielding. The provisions in this section have changed little since 1963. They are similar to the provisions in Section A-G2 in the 1999 *LRFD Specification* (AISC, 2000b), and similar to the provisions in Section G2 in the 1989 *ASD Specification* (AISC, 1989). The provisions for plate girders are based on research reported in Basler and Thurlimann (1963).

There is no seamless transition between the equations in Section F4 and F5. Thus the bending strength of a girder with $F_y = 50$ ksi (345 MPa) and a web slenderness $h/t_w = 137$ is not close to that of a girder with $h/t_w = 138$. These two slenderness ratios are on either side of the limiting ratio. This gap is caused by the discontinuity between the lateral-torsional buckling resistances predicted by Section F4 and those predicted by Section F5 due to the implicit use of $J = 0$ in Section F5. However, for typical noncompact web section members close to the noncompact web limit, the influence of J on the lateral-torsional buckling resistance is relatively small (for example, the calculated L_r values including J versus using $J = 0$ typically differ by less than 10 percent). The implicit use of $J = 0$ in Section F5 is intended to account for the influence of web distortional flexibility on the lateral-torsional buckling resistance for slender-web I-section members.

F6. I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MINOR AXIS

I-shaped members and channels bent about their minor axis do not experience lateral-torsional buckling or web buckling. The only limit states to consider are yielding and flange local buckling. The user note informs the designer of the few rolled shapes that need to be checked for flange local buckling.

F7. SQUARE AND RECTANGULAR HSS AND BOX-SHAPED MEMBERS

The provisions for the nominal flexural strength of HSS include the limit states of yielding and local buckling. Square and rectangular HSS bent about the minor axis are not subject to lateral-torsional buckling.

Because of the high torsional resistance of the closed cross-section, the critical unbraced lengths L_p and L_r that correspond to the development of the plastic moment and the yield moment, respectively, are very large. For example, as shown in Figure C-F7.1, an HSS 20 × 4 × 5/16 (HSS 508 × 101.6 × 7.9), which has one of the largest depth-width ratios among standard HSS, has L_p of 6.7 ft (2.0 m) and L_r of 137 ft (42 m) as determined in accordance with the 1993 *LRFD Specification* (AISC, 1993). An extreme deflection limit might correspond to a length-to-depth ratio of 24 or a length of 40 ft (12 m) for this member. Using the specified linear reduction between the plastic moment and the yield moment for lateral-torsional buckling, the plastic moment is reduced by only 7 percent for the 40-ft (12 m) length. In most practical designs where the moment gradient C_b is larger than unity, the reduction will be nonexistent or insignificant.

The provisions for local buckling of noncompact rectangular HSS are also the same as those in the previous sections of this chapter: $M_n = M_p$ for $b/t \le \lambda_p$, and a linear transition from M_p to $F_y S_x$ when $\lambda_p < b/t \le \lambda_r$. The equation for the effective width of the compression flange when b/t exceeds λ_r is the same as that used for rectangular HSS in axial compression except that the stress is taken as the yield stress. This implies that the stress in the corners of the compression flange is at yield when the ultimate post-buckling strength of the flange is reached. When using the effective width, the nominal flexural strength is determined from the effective section modulus to the compression flange using the distance from the shifted neutral axis. A slightly conservative estimate of the nominal flexural strength can be obtained by using the effective width for both the compression and tension flange, thereby maintaining the symmetry of the cross section and simplifying the calculations.

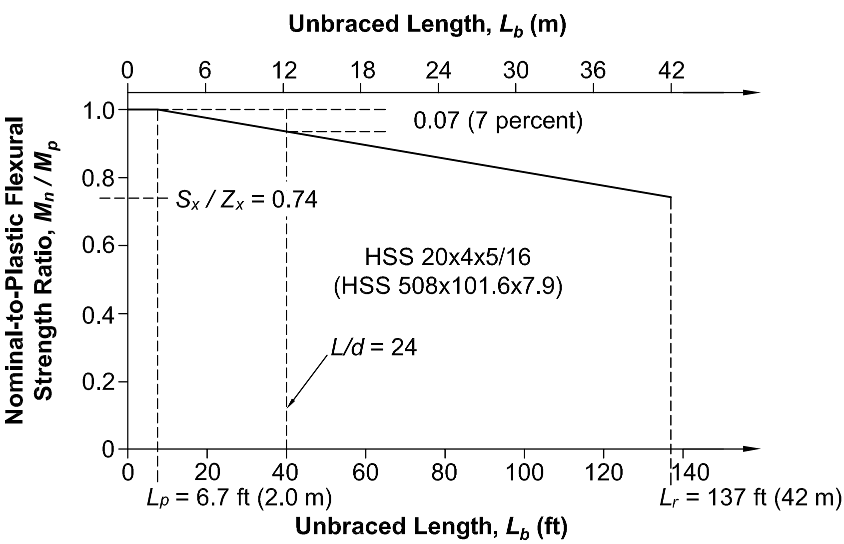


Fig. C-F7.1. Lateral-torsional buckling of rectangular HSS.

F8. ROUND HSS

Round HSS are not subject to lateral-torsional buckling. The failure modes and post-buckling behavior of round HSS can be grouped into three categories (Sherman, 1992; Galambos, 1998):

(a) For low values of D/t, a long plastic plateau occurs in the moment-rotation curve. The cross section gradually ovalizes, local wave buckles eventually form, and the moment resistance subsequently decays slowly. Flexural strength may exceed the theoretical plastic moment due to *strain hardening*.
(b) For intermediate values of D/t, the plastic moment is nearly achieved but a single local buckle develops and the flexural strength decays slowly with little or no plastic plateau region.
(c) For high values of D/t HSS, multiple buckles form suddenly with very little ovalization and the flexural strength drops quickly.

The flexural strength provisions for round HSS reflect these three regions of behavior and are based upon five experimental programs involving hot-formed seamless pipe, electric-resistance-welded pipe and fabricated tubing (Galambos, 1998).

F9. TEES AND DOUBLE ANGLES LOADED IN THE PLANE OF SYMMETRY

The lateral-torsional buckling (LTB) strength of singly symmetric tee beams is given by a fairly complex formula (Galambos, 1998). Equation F9-4 is a simplified formulation based on Kitipornchai and Trahair (1980). See also Ellifritt, Wine, Sputo, and Samuel (1992).

The C_b factor used for I-shaped beams is unconservative for tee beams with the stem in compression. For such cases $C_b = 1.0$ is appropriate. When beams are bent in reverse curvature, the portion with the stem in compression may control the LTB resistance even though the moments may be small relative to other portions of the unbraced length with $C_b \approx 1.0$. This is because the LTB strength of a tee with the stem in compression may be only about one-fourth of the strength for the stem in tension. Since the buckling strength is sensitive to the moment diagram, C_b has been conservatively taken as 1.0. In cases where the stem is in tension, connection details should be designed to minimize any end restraining moments that might cause the stem to be in compression.

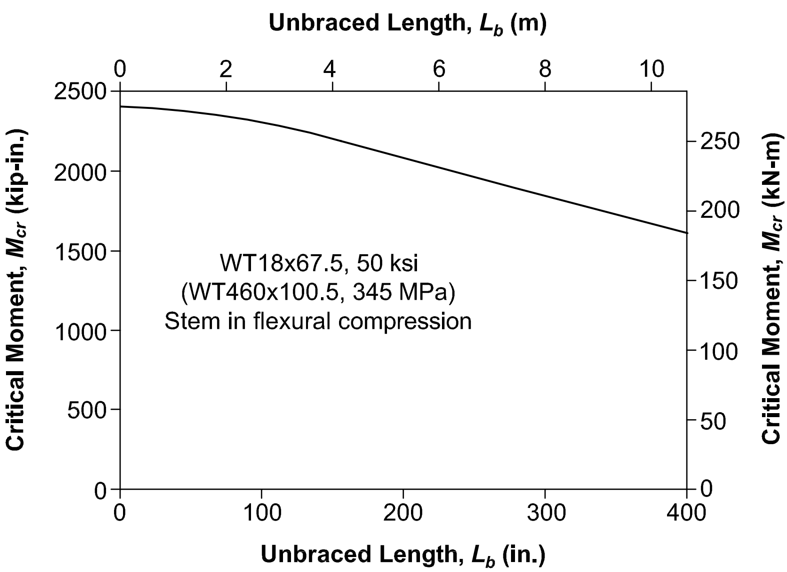
No limiting stem width-thickness ratio, λ_r, is provided in this section to account for the local buckling of the stem when it is in compression. The reason for this omission is that the lateral-torsional buckling equations (Equations F9-4 and F9-5) also give the local buckling strength as L_b approaches zero. This is not immediately evident, because when $L_b = 0$ is substituted into these equations one obtains, after some algebraic manipulations, $M_{cr} = 0/0$, which is a mathematically indeterminate expression. From elementary calculus such a problem is solved by differentiating the numerator and the denominator as often as needed to arrive at an explicit expression using L'Hospital's rule. If this operation is performed twice, one can obtain the following equation for the critical moment of combined lateral-torsional and local buckling:

$$M_{cr,L_b=0} = \frac{\pi EJ\sqrt{\dfrac{G}{E}}}{4.6d} = 0.424\frac{EJ}{d} \tag{C-F9-1}$$

The relationship between the unbraced length and the critical moment for a WT18×67.5 (WT460×100.5) [$F_y = 50$ ksi (345 MPa)] tee beam, with the stem in flexural compression, is shown in Figure C-F9.1.

Although flexure about the y-axis of tees and double angles does not occur frequently, guidance is given here to address this condition. The yield limit state and the local buckling limit state of the flange can be checked by using Equations F6-1 through F6-4. Lateral-torsional buckling can conservatively be calculated by assuming the flange acts alone as a rectangular beam, using Equations F11-2 through F11-4. Alternately an elastic critical moment given as

$$M_e = \frac{\pi}{L_b}\sqrt{EI_xGJ} \tag{C-F9-2}$$



Critical moment when $L_b = 0$: M_{LB} = 2409 kip-in. (274.25 kN-m)
This is also local buckling of the stem under flexural compression.
Yield moment: M_y = 2485 kip-in. (282.90 kN-m)

Fig. C-F9.1. Critical moment for a tee beam [WT18×67.5 (WT460×100.5), $F_y = 50$ ksi (345 MPa)].

may be used in Equations F10-2 or F10-3 to obtain the nominal flexural strength.

F10. SINGLE ANGLES

Flexural strength limits are established for the limit states of yielding, local buckling and lateral-torsional buckling of single-angle beams. In addition to addressing the general case of unequal-leg single angles, the equal-leg angle is treated as a special case. Furthermore, bending of equal-leg angles about a geometric axis, an axis parallel to one of the legs, is addressed separately as it is a common case of angle bending.

The tips of an angle refer to the free edges of the two legs. In most cases of unrestrained bending, the flexural stresses at the two tips will have the same sign (tension or compression). For constrained bending about a geometric axis, the tip stresses will differ in sign. Provisions for both tension and compression at the tip should be checked as appropriate, but in most cases it will be evident which controls.

Appropriate serviceability limits for single-angle beams need also to be considered. In particular, for longer members subjected to unrestrained bending, deflections are likely to control rather than lateral-torsional or local buckling strength.

The provisions in this section follow the general format for nominal flexural resistance (see Figure C-F1.2). There is a region of full yielding, a linear transition to the yield moment, and a region of local buckling.

1. Yielding

The strength at full yielding is limited to a shape factor of 1.50 applied to the yield moment. This leads to a lower bound plastic moment for an angle that could be bent about any axis, inasmuch as these provisions are applicable to all flexural conditions. The 1.25 factor originally used was known to be a conservative value. Recent research work (Earls and Galambos, 1997) has indicated that the 1.50 factor represents a better lower bound value. Since the shape factor for angles is in excess of 1.50, the nominal design strength $M_n = 1.5M_y$ for compact members is justified provided that instability does not control.

2. Lateral-Torsional Buckling

Lateral-torsional buckling may limit the flexural strength of an unbraced single-angle beam. As illustrated in Figure C-F10.1, Equation F10-2 represents the elastic buckling portion with the maximum nominal flexural strength, M_n, equal to 75 percent of the theoretical buckling moment, M_e. Equation F10-3 represents the inelastic buckling transition expression between $0.75M_y$ and $1.5M_y$. The maximum beam flexural strength $M_n = 1.5M_y$ will occur when the theoretical buckling moment, M_e, reaches or exceeds $7.7M_y$. These equations are modifications of those developed from the results of Australian research on single angles in flexure and on an analytical model consisting of two rectangular elements of length equal to the actual angle leg width minus one-half the thickness (AISC, 1975; Leigh and Lay, 1978; Leigh and Lay, 1984; Madugula and Kennedy, 1985).

When bending is applied about one leg of a laterally unrestrained single angle, the angle will deflect laterally as well as in the bending direction. Its behavior can be evaluated by resolving the load and/or moments into principal axis components and determining the sum of these principal axis flexural effects. Section F10.2(i) is provided to simplify and expedite the calculations for this common situation with equal-leg angles.

For such unrestrained bending of an equal-leg angle, the resulting maximum normal stress at the angle tip (in the direction of bending) will be approximately 25 percent greater than the calculated stress using the geometric axis section modulus. The value of M_e in Equation F10-5 and the evaluation of M_y using 0.80 of the geometric axis section modulus reflect bending about the inclined axis shown in Figure C-F10.2.

The deflection calculated using the geometric axis moment of inertia has to be increased 82 percent to approximate the total deflection. Deflection has two components, a vertical component (in the direction of applied load) 1.56 times the calculated value and a horizontal component of 0.94 times the calculated value. The resultant total deflection is in the general direction of the weak principal axis bending of the angle (see Figure C-F10.2). These unrestrained bending deflections should be considered in evaluating serviceability and will often control the design over lateral-torsional buckling.

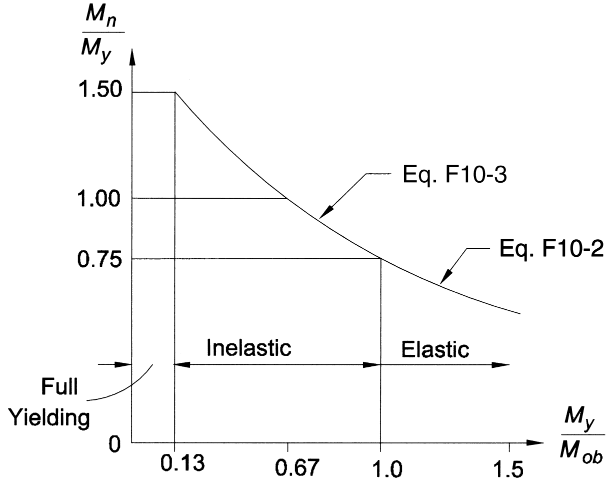


Fig. C-F10.1. Lateral-torsional buckling limits of a single-angle beam.

The horizontal component of deflection being approximately 60 percent of the vertical deflection means that the lateral restraining force required to achieve purely vertical deflection must be 60 percent of the applied load value (or produce a moment 60 percent of the applied value) which is very significant.

Lateral-torsional buckling is limited by M_e (Leigh and Lay, 1978; Leigh and Lay, 1984) in Equation F10-4a, which is based on

$$M_{cr} = \frac{2.33Eb^4t}{(1+3\cos^2\theta)(Kl)^2} \times \left[\sqrt{\sin^2\theta + \frac{0.156(1+3\cos^2\theta)(Kl)^2t^2}{b^4}} + \sin\theta\right] \tag{C-F10-1}$$

(the general expression for the critical moment of an equal-leg angle) with $\theta = 45°$ or the condition where the angle tip stress is compressive (see Figure C-F10.3). Lateral-torsional buckling can also limit the flexural strength of the cross section when the maximum angle tip stress is tensile from geometric axis flexure, especially with use of the flexural strength limits in Section F10.2. Using $\theta = 45°$ in Equation C-F10-1, the resulting expression is Equation F10-4b with a $+1$ instead of -1 as the last term.

Stress at the tip of the angle leg parallel to the applied bending axis is of the same sign as the maximum stress at the tip of the other leg when the single angle is unrestrained. For an equal-leg angle this stress is about one-third of the maximum stress. It is only necessary to check the nominal bending strength based on the tip of the angle leg with the maximum stress when evaluating such an angle. Since this maximum moment per Section F10.2(ii) represents combined principal axis moments and Equation F10-5 represents the design limit for these

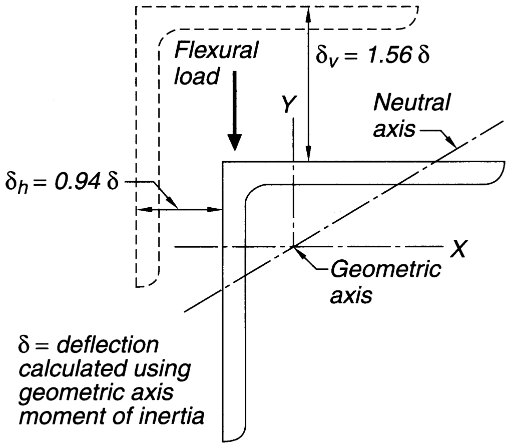


Fig. C-F10.2. Geometric axis bending of laterally unrestrained equal-leg angles.

combined flexural moments, only a single flexural term needs to be considered when evaluating combined flexural and axial effects.

For unequal-leg angles without lateral-torsional restraint, the applied load or moment must be resolved into components along the two principal axes in all cases and design must be for *biaxial bending* using the interaction equations in Chapter H.

Under major axis bending of equal-leg angles, Equation F10-5 in combination with Equations F10-2 and F10-3 controls the available moment against overall lateral-torsional buckling of the angle. This is based on M_e, given earlier with $\theta = 0$.

Lateral-torsional buckling for this case will reduce the stress below $1.5M_y$ only for $l/t \geq 3675C_b/F_y(M_e = 7.7M_y)$. If the lt/b^2 parameter is small (less than approximately $0.87C_b$ for this case), local buckling will control the available moment and M_n based on lateral-torsional buckling need not be evaluated. Local buckling must be checked using Section F10.3.

Lateral-torsional buckling about the major principal w-axis of an unequal-leg angle is controlled by M_e in Equation F10-6. The section property β_w reflects the location of the shear center relative to the principal axis of the section and the bending direction under uniform bending. Positive β_w and maximum M_e occurs when the shear center is in flexural compression while negative β_w and minimum M_e occur when the shear center is in flexural tension (see Figure C-F10.4). This β_w effect is consistent with behavior of singly symmetric I-shaped beams, which are more stable when the compression flange is larger than the tension flange. For principal w-axis bending of equal-leg angles, β_w is equal to zero due to symmetry and Equation F10-6 reduces to Equation F10-5 for this special case.

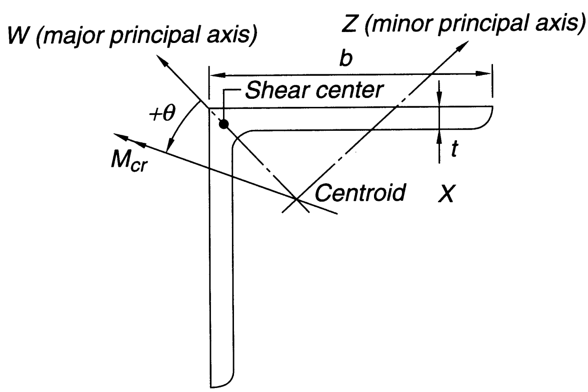


Fig. C-F10.3. Equal-leg angle with general moment loading.

For reverse curvature bending, part of the unbraced length has positive β_w, while the remainder has negative β_w; conservatively, the negative value is assigned for that entire unbraced segment.

The factor β_w is essentially independent of angle thickness (less than one percent variation from mean value) and is primarily a function of the leg widths. The average values shown in Table C-F10.1 may be used for design.

3. Leg Local Buckling

The b/t limits have been modified to be more representative of flexural limits rather than using those for single angles under uniform compression. Typically the flexural stresses will vary along the leg length permitting the use of the stress limits given. Even for the geometric axis flexure case, which produces uniform compression along one leg, use of these limits will provide a conservative value when compared to the results reported in Earls and Galambos (1997).

F11. RECTANGULAR BARS AND ROUNDS

The provisions in Section F11 apply to solid bars with round and rectangular cross section. The prevalent limit state for such members is the attainment of the full plastic moment, M_p. The exception is the lateral-torsional buckling of rectangular bars where the depth is larger than the width. The requirements for design are identical to those given previously in Table A-F1.1 in the 1999 *LRFD Specification* (AISC, 2000b). Since the shape factor for a rectangular cross section is 1.5 and for a round section is 1.7, consideration must be given to serviceability issues such as excessive deflection or permanent deformation under service-load conditions.

F12. UNSYMMETRICAL SHAPES

When the design engineer encounters beams that do not contain an axis of symmetry, or any other shape for which there are no provisions in the other sections of Chapter F, the stresses are to be limited by the yield stress or the elastic buckling

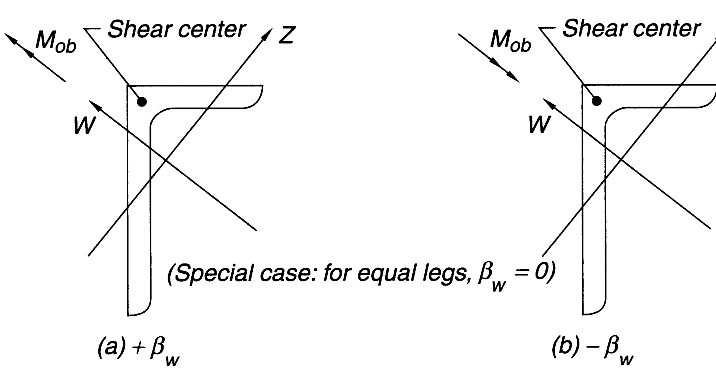


Fig. C-F10.4. Unequal-leg angle in bending.

TABLE C-F10.1
β_w Values for Angles

Angle Size [in. (mm)]	β_w [in. (mm)]*
9 × 4 (229 × 102)	6.54 (116)
8 × 6 (203 × 152) 8 × 4 (203 × 102)	3.31 (84.1) 5.48 (139)
7 × 4 (178 × 102)	4.37 (111)
6 × 4 (152 × 102) 6 × 3.5 (152 × 89)	3.14 (79.8) 3.69 (93.7)
5 × 3.5 (127 × 89) 5 × 3 (127 × 76)	2.40 (61.0) 2.99 (75.9)
4 × 3.5 (102 × 89) 4 × 3 (102 × 76)	0.87 (22.1) 1.65 (41.9)
3.5 × 3 (89 × 76) 3.5 × 2.5 (89 × 64)	0.87 (22.1) 1.62 (41.1)
3 × 2.5 (76 × 64) 3 × 2 (76 × 51)	0.86 (21.8) 1.56 (39.6)
2.5 × 2 (64 × 51)	0.85 (21.6)
Equal legs	0.00

*$\beta_w = \frac{1}{I_w}\int_A z\left(w^2 + z^2\right)dA - 2z_o$ where z_o is the coordinate along the z-axis of the shear center with respect to the centroid, and I_w is the moment of inertia for the major principal axis; β_w has positive or negative value depending on direction of bending (see Figure C-F10.4)

stress. The stress distribution and/or the elastic buckling stress must be determined from principles of structural mechanics, text books or handbooks, such as the SSRC Guide (Galambos, 1998), papers in journals, or finite element analyses. Alternatively, the designer can avoid the problem by selecting cross sections from among the many choices given in the previous sections of Chapter F.

F13. PROPORTIONS OF BEAMS AND GIRDERS

1. Hole Reductions

Historically, provisions for proportions of rolled beams and girders with holes in the tension flange were based upon either a percentage reduction independent of material strength or a calculated relationship between the tension rupture and tension yield strengths of the flange, with resistance factors or safety factors included in the calculation. In both cases, the provisions were developed based upon tests of steel with a specified minimum yield stress of 36 ksi (248 MPa) or less.

More recent tests (Dexter and Altstadt, 2004; Yuan, Swanson, and Rassati, 2004) indicate that the flexural strength on the net section is better predicted by comparison of the quantities $F_y A_{fg}$ and $F_u A_{fn}$, with slight adjustment when the ratio of F_y to F_u exceeds 0.8. If the holes remove enough material to affect the member strength, the critical stress is adjusted from F_y to $(F_u A_{fn}/A_{fg})$ and this value is conservatively applied to the elastic section modulus S_x.

2. Proportioning Limits for I-Shaped Members

The provisions of this section are taken directly from Appendix G, Section G1 of the 1999 *LRFD Specification* (AISC, 2000b). They have been part of the plate-girder design requirements since 1963; they are derived from Basler and Thurlimann (1963). The web depth-thickness limitations are provided so as to prevent the flange from buckling into the web. Equation F13-4 is slightly modified from the corresponding Equation A-G1-2 in the 1999 Specification to recognize the change in this Specification in the definition of *residual stress* from a flat 16.5 ksi (114 MPa) used previously to 30 percent of the yield stress, as shown by the following derivation,

$$\frac{0.48E}{\sqrt{F_y\left(F_y+16.5\right)}} \approx \frac{0.48E}{\sqrt{F_y\left(F_y+0.3F_y\right)}} = \frac{0.42E}{F_y} \qquad \text{(C-F13-1)}$$

CHAPTER G

DESIGN OF MEMBERS FOR SHEAR

G1. GENERAL PROVISIONS

Chapter G applies to webs of singly or doubly symmetric members subject to shear in the plane of the web, single angles and HSS, and shear in the weak direction of singly or doubly symmetric shapes.

Two methods for determining the shear strength of singly or doubly symmetric I-shaped beams and built-up sections are presented. The method of Section G2 does not utilize the post-buckling strength of the web, while the method of Section G3 utilizes the post-buckling strength.

G2. MEMBERS WITH UNSTIFFENED OR STIFFENED WEBS

Section G2 deals with the shear strength of webs of wide-flange or I-shaped members, as well as webs of tee-shapes, that are subject to shear and bending in the plane of the web. The provisions in Section G2 apply to the general case when an increase of strength due to tension field action is not permitted. Conservatively, these provisions may be applied also when it is not desired to use the tension field action enhancement for convenience in design. Consideration of the effect of bending on the shear strength is not required because the effect is deemed negligible.

1. Nominal Shear Strength

The nominal shear strength of a web is defined by Equation G2-1, a product of the shear yield force $0.6F_yA_w$ and the shear-buckling reduction factor C_v.

The provisions of case (a) in Section G2.1 for rolled I-shaped members with $h/t_w \leq 2.24\sqrt{E/F_y}$ are similar to previous LRFD provisions, with the exception that ϕ has been increased from 0.90 to 1.00 (with a corresponding decrease of the safety factor from 1.67 to 1.5), thus making these provisions consistent with previous provisions for allowable stress design. The value of ϕ of 0.90 is justified by comparison with experimental test data and recognizes the minor consequences of shear yielding, as compared to those associated with tension and compression yielding, on the overall performance of rolled I-shaped members. This increase is applicable only to the shear yielding limit state of I-shaped members.

Case (b) in Section G2.1 uses the shear buckling reduction factor, C_v, shown in Figure C-G2.1. The curve for C_v has three segments.

For webs with $h/t_w \le 1.10\sqrt{Ek_v/F_y}$, the nominal shear strength V_n is based on shear yielding of the web, with C_v given by Equation G2-3. This h/t_w limit was determined by setting the critical stress causing shear buckling, F_{cr}, equal to the yield stress of the web, $F_{yw} = F_y$, in Equation 35 of Cooper, Galambos, and Ravindra (1978).

When $h/t_w > 1.10\sqrt{Ek_v/F_y}$, the web shear strength is based on buckling. It has been suggested to take the proportional limit as 80 percent of the yield stress of the web (Basler, 1961). This corresponds to $h/t_w = (1.10/0.8)\left(\sqrt{Ek_v/F_y}\right)$.

When $h/t_w > 1.37\sqrt{Ek_v/F_y}$, the web strength is determined from the elastic buckling stress given by Equation 6 of Cooper and others (1978) and Equation 9-7 in Timoshenko and Gere (1961):

$$F_{cr} = \frac{\pi^2 E k_v}{12\left(1 - \nu^2\right)\left(h/t_w\right)^2} \tag{C-G2-1}$$

C_v in Equation G2-5 was obtained by dividing F_{cr} from Equation C-G2-1 by $0.6F_y A_w$ and using $E = 29{,}000$ ksi (200 000 MPa) and $\nu = 0.3$.

A straight-line transition for C_v (Equation G2-4) is used between the limits given by $1.10\sqrt{k_v E/F_y} < h/t_w \le 1.37\sqrt{k_v E/F_y}$.

The plate buckling coefficient, k_v, for panels subject to pure shear having simple supports on all four sides is given by Equation 4.24 in Galambos (1998).

$$k_v = \left\{ \begin{array}{ll} 4.00 + \dfrac{5.34}{(a/h)^2} & \text{for } a/h \le 1 \\ 5.34 + \dfrac{4.00}{(a/h)^2} & \text{for } a/h > 1 \end{array} \right\} \tag{C-G2-2}$$

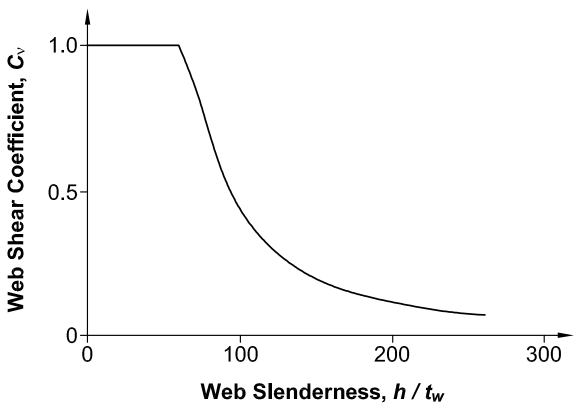


Fig. C-G2.1. Shear buckling coefficient C_v for $F_y = 50$ ksi (345 MPa) and $k_v = 5.0$.

For practical purposes and without loss of accuracy, these equations have been simplified herein and in AASHTO (1998) to

$$k_v = 5 + \frac{5}{(a/h)^2} \quad \text{(C-G2-3)}$$

When the panel ratio a/h becomes large, as in the case of webs without transverse stiffeners, then $k_v = 5$. Equation C-G2-3 applies as long as there are flanges on both edges of the web. For tee-shaped beams the free edge is unrestrained and for this situation $k_v = 1.2$ (JCRC, 1971).

The provisions of Section G2.1 assume monotonically increasing loads. If a flexural member is subjected to load reversals causing cyclic yielding over large portions of a web, such as may occur during a major earthquake, special design considerations may apply (Popov, 1980).

2. Transverse Stiffeners

When transverse stiffeners are needed, they must be rigid enough to cause a buckling node to form along the line of the stiffener. This requirement applies whether or not tension field action is counted upon. The required moment of inertia of the stiffener is the same as in AASHTO (1996), but it is different from the formula $I_{st} \geq (h/50)^4$ in the 1989 *ASD Specification* (AISC, 1989). Equation G2-5 is derived in Chapter 11 of Salmon and Johnson (1996). The origin of the formula can be traced to Bleich (1952).

G3. TENSION FIELD ACTION

The provisions of Section G3 apply when it is intended to account for the enhanced strength of webs of built-up members due to tension field action.

1. Limits on the Use of Tension Field Action

The panels of the web of a built-up member, bounded on top and bottom by the flanges and on each side by the transverse stiffeners, are capable of carrying loads far in excess of their "web buckling" load. Upon reaching the theoretical web buckling limit, very slight lateral web displacements will have developed. These deformations are of no structural significance, because other means are still present to provide further strength.

When transverse stiffeners are properly spaced and are strong enough to act as compression struts, membrane stresses due to shear forces greater than those associated with the theoretical web buckling load form diagonal tension fields in the web panels. The resulting combination in effect provides a Pratt truss that furnishes the strength to resist applied shear forces unaccounted for by the linear buckling theory.

The key point in the development of tension field action in the web of plate girders is the ability of the stiffeners to support the compression from the two panels on either side of the stiffener. In the case of end panels there is a panel only on one side. The support of the tension field forces is also reduced when the panel aspect ratio becomes too large. For this reason the inclusion of the tension field enhancement is not permitted for end panels and when *a/h* exceeds 3.0 or $\left[\dfrac{260}{(h/t_w)}\right]^2$.

2. Nominal Shear Strength with Tension Field Action

Analytical methods based on tension field action have been developed (Basler and Thurlimann, 1963; Basler, 1961) and corroborated in an extensive program of tests (Basler, Yen, Mueller, and Thurlimann, 1960). Equation G3-2 is based on this research. The second term in the bracket represents the relative increase of the panel shear strength due to tension field action.

3. Transverse Stiffeners

The vertical component of the tension field force that is developed in the web panel must be resisted by the transverse stiffener. In addition to the rigidity required to keep the line of the stiffener as a nonmoving point for the buckled panel, as provided for in Section G2.2, the stiffener must also have a large enough area to resist the tension field reaction. Equation G3-3 often controls the design of the stiffeners.

G4. SINGLE ANGLES

Shear stresses in single-angle members are the result of the gradient of the bending moment along the length (flexural shear) and the torsional moment.

The maximum elastic stress due to flexural shear is

$$f_v = \frac{1.5V_b}{bt} \tag{C-G4-1}$$

where V_b is the component of the shear force parallel to the angle leg with width b and thickness t. The stress is constant throughout the thickness, and it should be calculated for both legs to determine the maximum. The coefficient 1.5 is the calculated value for equal leg angles loaded along one of the principal axes. For equal leg angles loaded along one of the geometric axes, this factor is 1.35. Factors between these limits may be calculated conservatively from V_bQ/It to determine the maximum stress at the neutral axis. Alternatively, if only flexural shear is considered, a uniform flexural shear stress in the leg of V_b/bt may be used due to inelastic material behavior and stress redistribution.

If the angle is not laterally braced against twist, a torsional moment is produced equal to the applied transverse load times the perpendicular distance e to the shear center, which is at the point of intersection of the centerlines of the two legs. Torsional moments are resisted by two types of shear behavior: pure torsion (*St. Venant torsion*) and *warping torsion* [see Seaburg and Carter (1997)]. The shear stresses due to restrained warping are small compared to the *St. Venant torsion* (typically less than 20 percent) and they can be neglected for practical purposes. The applied torsional moment is then resisted by pure shear stresses that are constant along the width of the leg (except for localized regions at the toe of the leg), and the maximum value can be approximated by

$$f_v = \frac{M_T t}{J} = \frac{3M_T}{At} \tag{C-G4-2}$$

where

J = torsional constant (approximated by $\Sigma(bt^3/3)$ when precomputed value is unavailable)
A = angle cross-sectional area

For a study of the effects of warping, see Gjelsvik (1981). Torsional moments from laterally unrestrained transverse loads also produce warping normal stresses that are superimposed on the bending stresses. However, since the warping strength of single angles is relatively small, this additional bending effect, just like the warping shear effect, can be neglected for practical purposes.

G5. RECTANGULAR HSS AND BOX MEMBERS

The two webs of a closed-section rectangular cross section resist shear the same way as the single web of an I-shaped plate girder or wide-flange beam, and therefore, the provisions of Section G2 apply.

G6. ROUND HSS

Little information is available on round HSS subjected to transverse shear and the recommendations are based on provisions for local buckling of cylinders due to torsion. However, since torsion is generally constant along the member length and transverse shear usually has a gradient; it is recommended to take the critical stress for transverse shear as 1.3 times the critical stress for torsion (Brockenbrough and Johnston, 1981; Galambos, 1998). The torsion equations apply over the full length of the member, but for transverse shear it is reasonable to use the length between the points of maximum and zero shear force. Only thin HSS may require a reduction in the shear strength based upon first shear yield. Even in this case, shear will only govern the design of round HSS for the case of thin sections with short spans.

In the equation for the nominal shear strength, V_n, of round HSS, it is assumed that the shear stress at the neutral axis, calculated as VQ/Ib, is at F_{cr}. For a thin round section with radius R and thickness t, $I = \pi R^3 t$, $Q = 2R^2 t$ and $b = 2t$. This gives the stress at the centroid as $V/\pi Rt$, in which the denominator is recognized as half the area of the round HSS.

G7. WEAK AXIS SHEAR IN SINGLY AND DOUBLY SYMMETRIC SHAPES

The nominal shear strength of singly and doubly symmetric I-shapes is governed by the equations of Section G2 with the plate buckling coefficient equal to $k_v = 1.2$, the same as the web of a tee-shape. The maximum plate slenderness of all rolled shapes is $(b_f/2t_f) = 13.8$, and for $F_y = 100$ ksi (690 MPa) the value of $1.10\sqrt{\frac{k_v E}{F_y}} = 1.10\sqrt{\frac{1.2 \times 29000}{100}} = 20.5$. Thus $C_v = 1.0$ except for built-up shapes with very slender flanges.

G8. BEAMS AND GIRDERS WITH WEB OPENINGS

Web openings in structural floor members may be used to accommodate various mechanical, electrical and other systems. Strength limit states, including local buckling of the compression flange or of the web, local buckling or yielding of the tee-shaped compression zone above or below the opening, lateral buckling and moment-shear interaction, or serviceability may control the design of a flexural member with web openings. The location, size and number of openings are important and empirical limits for them have been identified. One general procedure for assessing these effects and the design of any needed reinforcement for both steel and composite beams is given in the ASCE *Specification for Structural Steel Beams with Web Openings* (ASCE, 1999), with background information provided in Darwin (1990) and in ASCE Task Committee on Design Criteria for Composite Structures in Steel and Concrete (1992) and ASCE Task Committee on Design Criteria for Composite Structures in Steel and Concrete (1992a).

CHAPTER H

DESIGN OF MEMBERS FOR COMBINED FORCES AND TORSION

Chapters D, E, F and G of this Specification address members subject to only one type of force: axial tension, axial compression, flexure and shear, respectively. Chapter H addresses members subject to a combination of two or more of the individual forces defined above, as well as possibly by additional forces due to torsion. The provisions fall into two categories: (a) the majority of the cases that can be handled by an interaction equation involving sums of ratios of required strengths to the available strengths; and (b) cases where the stresses due to the applied forces are added and compared to limiting buckling or yield stresses. Designers will have to consult the provisions of Sections H2 and H3 only in rarely occurring cases.

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

1. Doubly and Singly Symmetric Members in Flexure and Compression

Section H1 contains design provisions for prismatic members under combined flexure and compression and under combined flexure and tension for doubly and singly symmetric sections. The provisions of Section H1 apply typically to rolled wide-flange shapes, channels, tee-shapes, round, square and rectangular HSS, solid rounds, squares, rectangles or diamonds, and any of the many possible combinations of doubly or singly symmetric shapes fabricated from plates and/or shapes by welding or bolting. The interaction equations accommodate flexure about one or both principal axes as well as axial compression or tension.

In 1923, the first AISC Specification required that the stresses due to flexure and compression be added and that the sum not exceed the allowable value. An interaction equation appeared first in the 1936 Specification, stating "Members subject to both axial and bending stresses shall be so proportioned that the quantity $\frac{f_a}{F_a} + \frac{f_b}{F_b}$ shall not exceed unity," in which F_a and F_b are, respectively, the axial and flexural allowable stresses permitted by this Specification, and f_a and f_b are the corresponding stresses due to the axial force and the bending moment, respectively. This linear interaction equation was in force until the 1961 Specification, when it was modified to account for frame stability and for the P-δ effect, that is, the secondary bending between the ends of the members (Equation C-H1-1). The P-Δ effect, that is, the second-order bending moment due to story sway, was not accommodated.

$$\frac{f_a}{F_a} + \frac{C_m f_b}{\left(1 - \frac{f_a}{F'_e}\right) F_b} \le 1.0 \qquad \text{(C-H1-1)}$$

The allowable axial stress F_a is determined for an effective length that is larger than unity for moment frames. The term $\dfrac{1}{1-\dfrac{f_a}{F'_e}}$ is the amplification of the interspan moment due to member deflection multiplied by the axial force (the P-δ effect). C_m accounts for the effect of the moment gradient. This interaction equation has been part of all the subsequent editions of the AISC ASD Specifications since 1961.

A new approach to the interaction of flexural and axial forces was introduced in the 1986 AISC *LRFD Specification* (AISC, 1986). The following is an explanation of the thinking behind the interaction curves used. The equations

$$\begin{aligned} &\frac{P}{P_y} + \frac{8}{9}\frac{M_{pc}}{M_p} = 1 \quad \text{for } \frac{P_u}{P_y} \ge 0.2 \\ &\frac{P}{2P_y} + \frac{M_{pc}}{M_p} = 1 \quad \text{for } \frac{P_u}{P_y} < 0.2 \end{aligned} \qquad \text{(C-H1-2)}$$

define the lower-bound curve for the interaction of the nondimensional axial strength P/P_y and flexural strength M/M_p for compact wide-flange stub-columns bent about their x-axis. The cross section is assumed to be fully yielded in tension and compression. The symbol M_{pc} is the plastic moment strength of the cross section in the presence of an axial force P. The curve representing Equation C-H1-2 almost overlaps the analytically exact curve for the major-axis bending of a W8×31 (W200×46.1) cross section (see Figure C-H1.1). The equations for the exact yield capacity of a wide-flange shape are (ASCE, 1971):

$$\begin{aligned} &\text{for } 0 \le \frac{P}{P_y} \le \frac{t_w\left(d - 2t_f\right)}{A} \\ &\qquad \frac{M_{pc}}{M_p} = 1 - \frac{A^2\left(\frac{P}{P_y}\right)^2}{4t_w Z_x} \\ &\text{for } \frac{t_w\left(d - 2t_f\right)}{A} \le \frac{P}{P_y} \le 1 \\ &\qquad \frac{M_{pc}}{M_p} = \frac{A\left(1 - \frac{P}{P_y}\right)}{2Z_x}\left[d - \frac{A\left(1 - \frac{P}{P_y}\right)}{2b_f}\right] \end{aligned} \qquad \text{(C-H1-3)}$$

The equation approximating the average yield strength of wide-flange shapes is

$$\frac{M_{pc}}{M_p} = 1.18\left(1 - \frac{P}{P_y}\right) \le 1 \qquad \text{(C-H1-4)}$$

The curves in Figure C-H1.2 show the exact and approximate yield interaction curves for wide-flange shapes bent about the y-axis, and the exact curves for the solid rectangular and round shapes. It is evident that the lower-bound AISC interaction curves are very conservative for these shapes.

The idea of portraying the strength of stub beam-columns was extended to actual beam-columns with actual lengths by normalizing the required flexural strength, M_u, of the beam by the nominal strength of a beam without axial force, M_n, and the required axial strength, P_u, by the nominal strength of a column without bending moment, P_n. This rearrangement results in a translation and rotation of the original stub-column interaction curve, as seen in Figure C-H1.3.

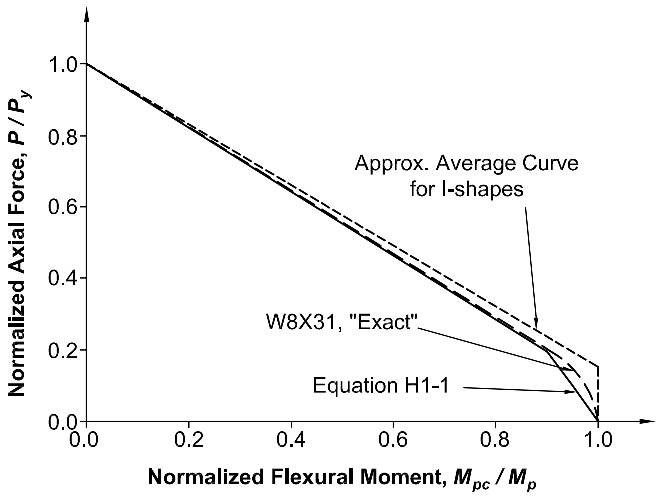


Fig. C-H1.1. Stub-column interaction curves: plastic moment versus axial force for wide-flange shapes, major-axis flexure [W8×31 (W200×46.1), F_y = 50 ksi (345 MPa)].

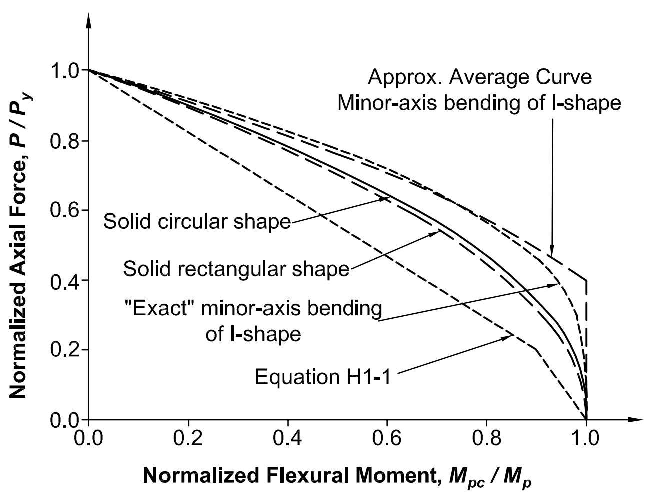


Fig. C-H1.2. Stub-column interaction curves: plastic moment versus axial force for solid round and rectangular sections and for wide-flange shapes, minor-axis flexure.

The normalized equations corresponding to the beam-column with length effects included are shown as Equation C-H1-5:

$$\frac{P_u}{P_n} + \frac{8}{9}\frac{M_u}{M_n} = 1 \text{ for } \frac{P_u}{P_n} \geq 0.2$$
$$\frac{P_u}{2P_n} + \frac{M_u}{M_n} = 1 \text{ for } \frac{P_u}{P_n} < 0.2 \qquad \text{(C-H1-5)}$$

The interaction equations are designed to be very versatile. The terms in the denominator fix the endpoints of the interaction curve. The nominal flexural strength, M_n, is determined by the appropriate provisions from Chapter F. It encompasses the limit states of yielding, lateral-torsional buckling, flange local buckling and web local buckling.

The axial term, P_n, is governed by the provisions of Chapter E, and it can accommodate compact or slender columns, as well as the limit states of major and minor axis buckling, and torsional and flexural-torsional buckling. Furthermore, P_n is calculated for the applicable effective length of the column to take care of frame stability effects, if the procedures of Section C.2-1a and Section C.2-1b are used to determine the required moments and axial forces. These moments and axial forces include the amplification due to second-order effects.

The utility of the interaction equations is further enhanced by the fact that they also permit the consideration of *biaxial bending*.

2. Doubly and Singly Symmetric Members in Flexure and Tension

Section H1.1 considers the most frequently occurring cases in design: members under flexure and axial compression. Section H1.2 addresses the less frequent cases of flexure and axial tension. Since axial tension increases the bending

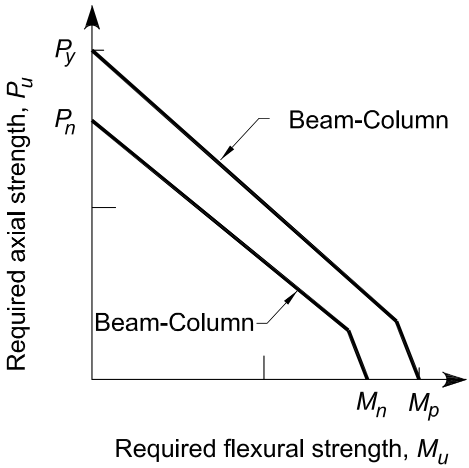


Fig. C-H1.3. Interaction curve for stub beam-column and beam-column.

stiffness of the member to some extent, Section H1.2 permits the increase of the bending terms in the interaction equations in proportion to $\sqrt{1+\dfrac{P_u}{P_{ey}}}$.

3. Doubly Symmetric Members in Single Axis Flexure and Compression

The linear interaction Equation C-H1-5 is conservative for cases where the axial limit state is out-of-plane buckling and the flexural limit state is lateral-torsional buckling for doubly symmetric wide-flange sections with moment applied about the x-axis (Galambos, 1998). Section H1.3 gives an optional equation for such beam-columns.

The two curves in Figure C-H1.4 illustrate the difference between the bi-linear and the parabolic interaction equations for the case of a W27×84 (W690×125) beam-column.

The relationship between Equations H1-1 and H1-2 is further illustrated in Figures C-H1.5 (for LRFD) and C-H1.6 (for ASD). The curves relate the required axial force, P (ordinate), and the required bending moment, M (abscissa), when the interaction Equations H1-1 and H1-2 are equal to unity. The positive values of P are compression and the negative values are tension. The curves are for a 10 ft (3 m) long W16×26 [$F_y = 50$ ksi (345 MPa)] member. The solid curve is for in-plane behavior, that is, lateral bracing prevents lateral-torsional buckling. The dotted curve represents Equation H1-1 for the case when there are no lateral braces between the ends of the beam-column. In the region of the tensile axial force, the curve is modified by the term $\sqrt{1+\dfrac{P}{P_y}}$, as permitted in Section H1.2. The dashed

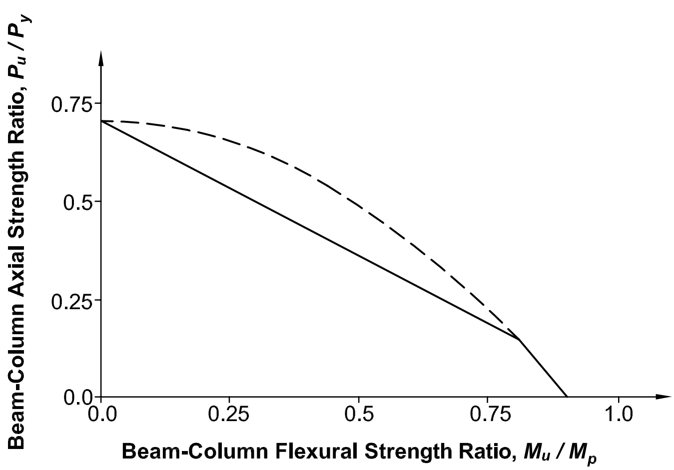


Fig. C-H1.4. Comparison between bi-linear (Equation H1-1) and parabolic (Equation H1-2) interaction equations [W27×84 (W690×125), $F_y = 50$ ksi (345 MPa), $L_b = 10$ ft (3.05 m), $C_b = 1.75$].

curve is Equation H1-2. For a given compressive or tensile axial force, the latter equation allows a larger bending moment over most of its domain of applicability.

H2. UNSYMMETRIC AND OTHER MEMBERS SUBJECT TO FLEXURE AND AXIAL FORCE

The provisions of Section H1 apply to beam-columns with cross sections that are either doubly or singly symmetric. However, there are many cross sections that are unsymmetrical, such as unequal leg angles and any number of possible fabricated sections. For these situations the interaction equation of Section H1 may not be appropriate. The linear interaction $\frac{f_a}{F_a} + \frac{f_{bw}}{F_{bw}} + \frac{f_{bz}}{F_{bz}} \leq 1.0$ provides a conservative and simple way to deal with such problems. The lower case stresses f are the required axial and flexural stresses computed by elastic analysis for the applicable loads, including second-order effects where appropriate, and the upper case stresses F are the available stresses corresponding to the limit state of yielding or buckling. The subscripts w and z refer to the principal axes of the unsymmetric cross section. This Specification leaves the option to the designer to use the Section H2 interaction equation for cross sections that would qualify for the more liberal interaction equation of Section H1.

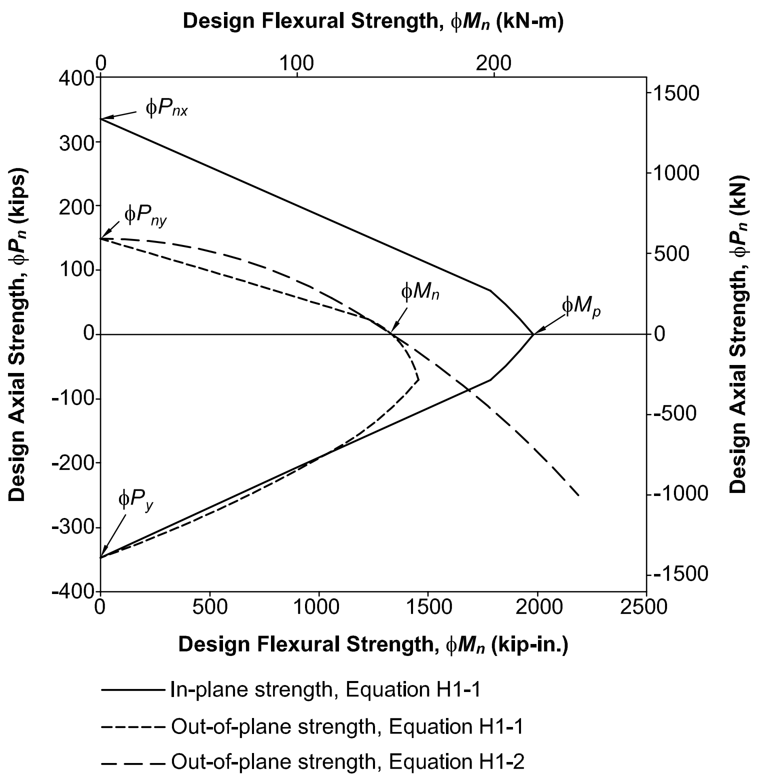


Fig. C-H1.5. Beam-columns under compressive and tensile axial force (tension is shown as negative) (LRFD) [W16×26 (W410×38.8), F_y = 50 ksi (345 MPa), L_b = 10 ft (3.05 m)].

The interaction equation, Equation H2-1, applies equally to the case where the axial force is in tension.

H3. MEMBERS UNDER TORSION AND COMBINED TORSION, FLEXURE, SHEAR AND/OR AXIAL FORCE

Section H3 provides provisions for cases not covered in the previous two sections. The first two parts of this section address the design of HSS members, and the third part is a general provision directed to cases where the designer encounters torsion in addition to normal stresses and shear stresses.

1. Torsional Strength of Round and Rectangular HSS

Hollow structural sections (HSS) are frequently used in space-frame construction and in other situations wherein significant torsional moments must be resisted by the members. Because of its closed cross section, an HSS is far more efficient in resisting torsion than an open cross section such as a W-shape or a channel. While normal and shear stresses due to restrained warping are usually significant in shapes of open cross section, they are insignificant in closed cross sections. The total torsional moment can be assumed to be resisted by pure torsional shear stresses. These are often referred in the literature as *St. Venant torsional* stresses.

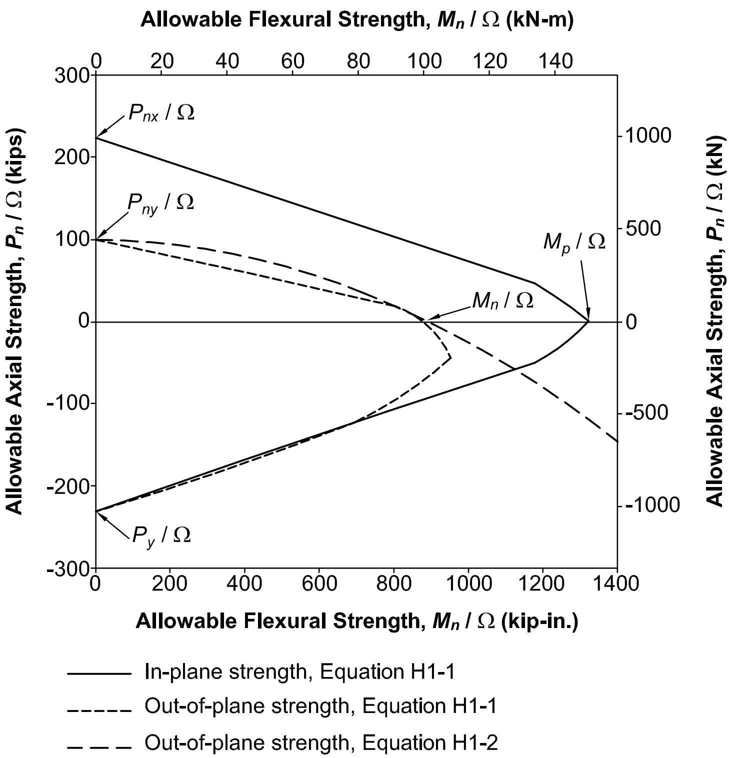


Fig. C-H1.6. Beam-columns under compressive and tensile axial force (tension is shown as negative) (ASD) [W16×26 (W410×38.8), $F_y = 50$ ksi (345 MPa), $L_b = 10$ ft (3.05 m)].

The pure torsional shear stress in HSS sections is assumed to be uniformly distributed along the wall of the cross section, and it is equal to the torsional moment, T_u, divided by a torsional shear constant for the cross section, C. In a limit state format, the nominal torsional resisting moment is the shear constant times the critical shear stress, F_{cr}.

For round HSS, the torsional shear constant is equal to the polar moment of inertia divided by the radius, which leads to

$$C = \frac{\pi t\,(D - t)^2}{2} \tag{C-H3-1}$$

For rectangular HSS, the torsional shear constant is obtained as $2tA_o$ using the membrane analogy (Timoshenko, 1956), where A_o is the area bounded by the midline of the section. Conservatively assuming an outside corner radius of $2t$, the midline radius is $1.5t$ and

$$A_o = t^2\,(B - t)\,(H - t)\,\frac{9\,(4 - \pi)}{4} \tag{C-H3-2}$$

resulting in

$$C = 2t\,(B - t)\,(H - t) - 4.5t^3\,(4 - \pi) \tag{C-H3-3}$$

The resistance factor ϕ and the safety factor Ω are the same as for flexural shear in Chapter G.

When considering local buckling in round HSS subjected to torsion, most structural members will either be long or of moderate length and the provisions for short cylinders will not apply. The elastic local buckling strength of long cylinders is unaffected by end conditions and the critical stress is given in Galambos (1998) as

$$F_{cr} = \frac{K_t E}{\left(\dfrac{D}{t}\right)^{\frac{3}{2}}} \tag{C-H3-4}$$

The theoretical value of K_t is 0.73 but a value of 0.6 is recommended to account for initial imperfections. An equation for the elastic local buckling stress for round HSS of moderate length ($L > 5.1D^2/t$) where the edges are not fixed at the ends against rotation is given in Schilling (1965) and Galambos (1998) as

$$F_{cr} = \frac{1.23E}{\left(\dfrac{D}{t}\right)^{\frac{5}{4}}\sqrt{\dfrac{L}{D}}} \tag{C-H3-5}$$

This equation includes a 15 percent reduction to account for initial imperfections. The length effect is included in this equation for simple end conditions, and the approximately 10 percent increase in buckling strength is neglected for edges fixed at the end. A limitation is provided so that the shear yield strength $0.6F_y$ is not exceeded.

The critical stress provisions for rectangular HSS are identical to the flexural shear provisions of Section G2 with the shear buckling coefficient equal to $k_v = 5.0$. The shear distribution due to torsion is uniform in the longest sides of a rectangular HSS, and this is the same distribution that is assumed to exist in the web of a W-shape beam. Therefore, it is reasonable that the provisions for buckling are the same in both cases.

2. HSS Subject to Combined Torsion, Shear, Flexure and Axial Force

Several interaction equation forms have been proposed in the literature for load combinations that produce both normal and shear stresses. In one common form, the normal and shear stresses are combined elliptically with the sum of the squares (Felton and Dobbs, 1967):

$$\left(\frac{f}{F_{cr}}\right)^2 + \left(\frac{f_v}{F_{vcr}}\right)^2 \le 1 \tag{C-H3-6}$$

In a second form, the first power of the ratio of the normal stresses is used:

$$\left(\frac{f}{F_{cr}}\right) + \left(\frac{f_v}{F_{vcr}}\right)^2 \le 1 \tag{C-H3-7}$$

The latter form is somewhat more conservative, but not overly so (Schilling, 1965), and this is the form used in this Specification:

$$\left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right) + \left(\frac{V_r}{V_c} + \frac{T_r}{T_c}\right)^2 \le 1.0 \tag{C-H3-8}$$

where the terms with the subscript r represent the required strengths, and the ones with the subscript c are the corresponding available strengths. Normal effects due to flexural and axial load effects are combined linearly and then combined with the square of the linear combination of flexural and torsional shear effects. When an axial compressive load effect is present, the required flexural strength, M_c, is to be determined by second-order analysis.

3. Strength of Non-HSS Members under Torsion and Combined Stress

This section covers all the cases not previously covered. Examples are built-up unsymmetric crane-girders and many other types of odd-shaped built-up cross sections. The required stresses are determined by elastic stress analysis based on established theories of structural mechanics. The three limit states to consider and the corresponding available stresses are:

(1) Yielding under normal stress—F_y
(2) Yielding under shear stress—$0.6F_y$
(3) Buckling—F_{cr}

In most cases it is sufficient to consider normal stresses and shear stresses separately because maximum values rarely occur in the same place in the cross section or at the same place in the span. Seaburg and Carter (1997) provides a complete discussion on torsional analysis of open shapes.

CHAPTER I

DESIGN OF COMPOSITE MEMBERS

Chapter I includes extensive technical and format changes as well as significant new material when compared to previous editions of the Specification. The major technical changes consist of new design provisions for composite columns (Section I2), which now include new cross-sectional strength models, provisions for tension and shear design, and a liberalization of the slenderness limits for HSS. Other significant technical changes have been made in the shear stud strength provisions (Section I3.2d): the use of an ultimate strength model for ASD design of composite beams (Section I3.2) and new material limitations (Section I1.2).

The main format changes in Chapter I include the elimination of the former Section I1, Design Assumptions and Definitions. The contents of that Section have been moved to the Glossary, the notation section, or other locations in the Specification and the section has been replaced by a section on General Provisions. Other format changes are as follows: the separation of composite column design into distinct provisions for concrete-encased sections and concrete-filled sections; and the incorporation of the former Section I5, Shear Connectors, into the current Section I3. In addition, the extensive historical notes on the development of composite design provisions present in the Commentary of the previous editions of the Specification have been eliminated as that material is now considered to be widely known.

I1. GENERAL PROVISIONS

Design of composite sections requires consideration of both steel and concrete behavior. These provisions were developed with the intent both to minimize conflicts between current steel and concrete design provisions and to give proper recognition to the advantages of composite design. As a result of the attempt to minimize conflicts, this Specification now uses a cross-sectional strength approach for column design consistent with that used in reinforced concrete design (ACI, 2002). This approach, in addition, results in a consistent treatment of cross-sectional strengths for both composite columns and beams.

This Specification assumes that the user is familiar with reinforced concrete design specifications such as ACI (2002) and does not repeat many of the provisions needed for the concrete portion of the design, such as material specifications, anchorage and splice lengths, and shear and torsion provisions.

The provisions in Chapter I address strength design of the composite sections only. The designer needs to consider the loads resisted by the steel section alone when determining load effects during the construction phase. The designer also needs to consider deformations throughout the life of the structure and the appropriate cross section for those deformations. When considering these latter limit states, due allowance should be made for the additional long-term changes in stresses and deformations due to creep and shrinkage of the concrete.

1. Nominal Strength of Composite Sections

The strength of composite sections shall be computed based on either of the two approaches presented in this Specification. The first is the strain compatibility approach, which provides a general calculation method. The second is the plastic stress distribution approach, which is a subset of the strain compatibility approach. The plastic stress distribution method provides a simple and convenient calculation method for the most common design situations, and is thus treated first.

1a. Plastic Stress Distribution Method

The plastic stress distribution method is based on the assumption of linear strain across the cross section and elasto-plastic behavior. It assumes that the concrete has reached its crushing strength in compression at a strain of 0.003 and a corresponding stress (typically $0.85 f_c'$) on a rectangular stress block, and that the steel has exceeded its yield strain, typically taken as F_y/E_s.

Based on these simple assumptions, the cross-sectional strength for different combinations of axial force and bending moment may be approximated, for typical composite column cross-sections. The actual interaction diagram for moment and axial force for a composite section based on a plastic stress distribution is similar to that of a reinforced concrete section as shown in Figure C-I1.1. As a simplification, for concrete-encased sections, a conservative linear interaction between four or five anchor points, depending on axis of bending, can be used (Roik and Bergmann, 1992; Galambos, 1998). These points are identified as A, B, C, D and E in Figure C-I1.1.

The plastic stress approach for columns assumes that no slip has occurred between the steel and concrete portions and that the required width-to-thickness ratios prevent local buckling from occurring until extensive yielding has taken place. Tests and analyses have shown that these are reasonable assumptions at the ultimate limit states for both concrete-encased steel sections with shear connectors and for

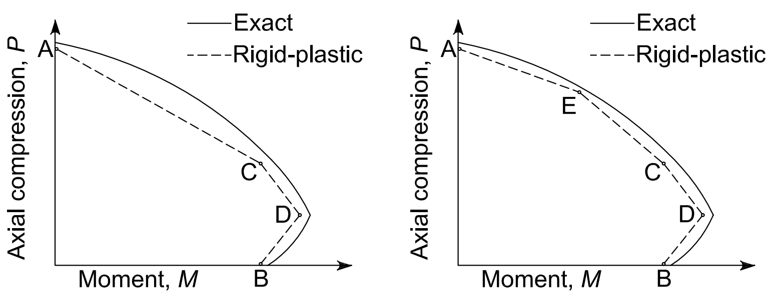


Fig. C-I1.1. Comparison between exact and simplified moment-axial compressive force envelopes.

HSS sections that comply with these provisions (Galambos, 1998; Hajjar, 2000; Shanmugam and Lakshmi, 2001). For circular HSS, these provisions allow for the increase of the usable concrete stress to $0.95 f'_c$ to account for the beneficial effects of the restraining hoop action arising from transverse confinement (Leon and Aho, 2002).

Based on similar assumptions, but allowing for slip between the steel beam and the composite slab, simplified expressions can also be derived for typical composite beam sections. Strictly speaking, these distributions are not based on slip, but on the strength of the shear connection. Full interaction is assumed if the shear connection strength exceeds that of either (a) the tensile yield strength of the steel section or the compressive strength of the concrete slab when the composite beam is loaded in positive moment, or (b) the tensile yield strength of the longitudinal reinforcing bars in the slab or the compressive strength of the steel section when loaded in negative moment. When shear connectors are provided in sufficient numbers to fully develop this flexural strength, any slip that occurs prior to yielding has a negligible affect on behavior. When full interaction is not present, the beam is said to be partially composite. The effects of slip on the elastic properties of a partially composite beam can be significant and should be accounted for, if significant, in calculations of deflections and stresses at service loads. Approximate elastic properties of partially composite beams are given in Commentary Section I3.

1b. Strain-Compatibility Approach

The principles used to calculate cross-sectional strength in Section I1.1a may not be applicable to all design situations or possible cross-sections. As an alternative, Section I1.1b permits the use of a generalized strain-compatibility approach that allows the use of any reasonable strain-stress model for the steel and concrete.

2. Material Limitations

The material limitations given in Section I1.2 reflect the range of material properties available from experimental testing (Galambos, 1998; Hajjar, 2000; Shanmugam and Lakshmi, 2001; Leon and Aho, 2002). As for reinforced concrete design, a limit of 10 ksi (70 MPa) is imposed for strength calculations, both to reflect the scant data available above this strength and the changes in behavior observed, particularly for brittle failure modes such as shear. A lower limit of 3 ksi (21 MPa) is specified for both normal and lightweight concrete and an upper limit of 6 ksi (42 MPa) is specified for lightweight concrete to encourage the use of good quality, yet readily available, grades of structural concrete. The use of higher strengths in computing the modulus of elasticity is permitted, and the limits given can be extended for strength calculations if appropriate testing and analyses are carried out.

3. Shear Connectors

This section provides basic shear connector details and material specifications. Nominal yield and tensile strengths of typical ASTM A108 Type B studs are 51 ksi (350 MPa) and 65 ksi (450 MPa), respectively (AWS 2004).

I2. AXIAL MEMBERS

In Section I2, the design of concrete-encased and concrete-filled composite columns is treated separately, although they have much in common. The intent is to facilitate design by keeping the general principles and detailing requirements for each type of column separate.

An ultimate strength cross-section model is used to determine the section strength (Leon and Aho, 2002). This model is similar to that used in previous LRFD Specifications. The major difference is that the full strength of the reinforcing steel and concrete are accounted for rather than the 70 percent that was used in those previous specifications. In addition, these provisions give the strength of the composite section as a force, while the previous approach had converted that force to an equivalent stress. Since the reinforcing steel and concrete had been arbitrarily discounted, the previous provisions did not accurately predict strength for columns with a low percentage of steel.

The design for length effects is consistent with that for steel columns. The equations used are the same as those in Chapter E, albeit in a slightly different format, and as the percent of concrete in the section decreases, the design defaults to that of a steel section. Comparisons between the provisions in the Specification and experimental data show that the method is generally conservative but that the coefficient of variation obtained is large (Leon and Aho, 2002).

1. Encased Composite Columns

1a. Limitations

(1) In this Specification, the use of composite columns is extended from the previous minimum steel ratio of 4 percent (area of steel shape divided by the gross area of the member) down to columns with a minimum of 1 percent. This is a direct result of using an ultimate strength cross-sectional approach, and removes the previous discontinuities in design that occurred as the steel ratio decreased below 4 percent.

(2) The specified minimum quantity for transverse reinforcement is intended to provide good confinement to the concrete.

(3) A minimum amount of longitudinal reinforcing steel is prescribed so that at least four continuous corner bars are used (see Section I2.1f). Other longitudinal bars may be needed to provide the required restraint to the cross-ties, but that longitudinal steel cannot be counted towards the cross-sectional strength unless it is continuous and properly anchored. It is expected that the limit will seldom be reached in practice, except for the case of a very large cross section.

1b. Compressive Strength

The compressive strength of the cross section is given as the sum of the ultimate strengths of the components. The strength is not capped as in reinforced concrete column design for a combination of the following reasons: (1) the resistance factor has been lowered from 0.85 in previous editions to 0.75 in this Specification; (2) the required transverse steel provides better performance than a typical reinforced concrete column; (3) the presence of a steel section near the center of the section reduces the possibility of a sudden failure due to buckling of the longitudinal reinforcing steel; and (4) in most cases there will be significant load eccentricities (in other words, moments) present due to the size of the member and the typical force introduction mechanisms.

1c. Tensile Strength

The new Section I2.1c has been added to clarify the tensile strength to be used in situations where uplift is a concern and for computations related to beam-column interaction. The provision focuses on the limit state of yield on gross area. Where appropriate for the structural configuration, consideration should also be given to other tensile strength and connection strength limit states as specified in Chapters D and J.

1d. Shear Strength

This new material has been added to provide guidance for the shear strength of composite columns. The provisions require either the use of the steel section alone plus the contribution from any transverse shear reinforcement present in the form of ties or the shear strength calculated based on the reinforced concrete portion of the cross-section alone (in other words, longitudinal and transverse reinforcing bars plus concrete). This implies the following shear strengths:

$$V_n = 0.6F_y A_w + A_{st} F_{yr} \frac{d}{s}$$

$$\phi = 0.9\,(\text{LRFD}) \qquad \Omega = 1.67\,(\text{ASD})$$

or

$$V_n = 2\sqrt{f_c'}\,bd + A_{st} F_{yr} \frac{d}{s}$$

$$\phi = 0.75\,(\text{LRFD}) \qquad \Omega = 2.00\,(\text{ASD})$$

It would be logical to suggest provisions where both the contributions of the steel section and the reinforced concrete are superimposed; however, there is little research available on this topic.

1e. Load Transfer

To avoid overstressing either the structural steel section or the concrete at connections in encased composite columns, a transfer of load by direct bearing, shear connection, or a combination of both is required. Although it is recognized that force transfer also occurs by direct bond interaction between the steel and concrete, this is typically ignored for encased composite columns (Griffis, 1992).

When shear connectors are used in encased composite columns, a uniform spacing is appropriate in most situations, but when large forces are applied, other connector arrangements may be needed to avoid overloading the component (steel section or concrete encasement) to which the load is applied directly.

When a supporting concrete area is wider on all sides than the loaded area, the nominal bearing strength for concrete may be taken as

$$N_b = 0.85 f_c' \sqrt{A_2/A_1} \tag{C-I2-1}$$

where A_1 is the loaded area and A_2 is the maximum area of the supporting surface that is geometrically similar and concentric with the loaded area. The value of $\sqrt{A_2/A_1}$ must be less than or equal to 2. This Specification uses the maximum nominal bearing strength of $1.7 f_c' A_B$. The resistance factor for bearing, ϕ_B, is 0.65 (and the associated safety factor Ω_B is 2.31) in accordance with ACI (2002).

2. Filled Composite Columns

2a. Limitations

(1) As discussed for encased columns, it is now permissible to design composite columns with a steel ratio as low as 1 percent.

(2) The specified minimum wall slenderness has been liberalized from previous editions of the *LRFD Specification*. Those editions did not differentiate between buckling of an unfilled and a filled HSS. The new provisions take into account the restraining effect of the concrete on the local buckling of the section wall.

2b. Compressive Strength

The compressive strength of the cross section is given as the sum of the ultimate strengths of the components. The beneficial confining effect of a circular HSS can be taken into account by increasing the crushing strength of the concrete to $0.95 f_c'$.

2c. Tensile Strength

As for encased columns, this new Section I2.2c has been added to clarify tensile strength.

2d. Shear Strength

See commentary to Section I2.1d.

2e. Load Transfer

To avoid overstressing either the structural steel section or the concrete at connections in filled composite columns, a transfer of load by direct bearing, shear connection, or direct bond interaction is permitted, with the mechanism providing the largest resistance being permissible for use. However, superposition of these force transfer mechanisms is not permitted for filled composite columns, as the experimental data indicate that direct bearing or shear connection often do not initiate until after direct bond interaction has been breached, and little experimental data is available about the interaction of direct bearing and shear interaction in filled composite columns.

Force transfer by direct bond is commonly used in filled composite columns as long as the connections are detailed to limit local deformations (API, 1993; Roeder and others, 1999). However, there is large scatter in the experimental data on the bond strength and associated force transfer length of filled composite columns, particularly when comparing tests in which the concrete core is pushed through the steel tube (push-out tests) to tests in which a beam is connected just to the steel tube and beam shear is transferred to the filled composite column. The added eccentricities of the connection tests typically raise the bond strength of the filled composite columns.

A reasonable lower bound value of bond strength of filled composite columns that meet the provisions of Section I2 is 60 psi (0.4 MPa). While push-out tests often show bond strengths below this value, eccentricity introduced into the connection is likely to increase the bond strength to this value or higher. Experiments also indicate that a reasonable assumption for the distance along the length of the filled composite column required to transfer the force from the steel HSS to the concrete core is approximately equal to the width of a rectangular HSS or the diameter of a round HSS, both above and below the point of load transfer.

One approach to estimating the direct bond interaction for filled HSS is presented below with recommendations for ϕ and Ω. These equations assume that one face of a rectangular filled composite column, or one-half of the perimeter of a circular filled composite column, is engaged in the transfer of stress by direct bond interaction. Higher values of nominal bond strength may be warranted for specific conditions. The scatter in the data leads to the recommended low value of the resistance factor, ϕ, and the corresponding high value of the safety factor, Ω.

(a) For rectangular HSS filled with concrete:

$$V_{in} = b^2 C_{in} F_{in} \tag{C-I2-2}$$

$$\phi = 0.45 \text{ (LRFD)} \qquad \Omega = 3.33 \text{ (ASD)}$$

where

V_{in} = nominal bond strength, kips (N)
F_{in} = nominal bond stress = 60 psi (0.40 MPa)
b = width of HSS along face transferring load, in. (mm)
C_{in} = 1 if the filled composite column extends only above or below the point of load transfer
= 2 if the filled composite column extends both above and below the point of load transfer

(b) For round HSS filled with concrete:

$$V_{in} = 0.5\pi D^2 C_{in} F_{in} \tag{C-I2-3}$$

$$\phi = 0.45 \text{ (LRFD)} \qquad \Omega = 3.33 \text{ (ASD)}$$

where

V_{in} = nominal bond strength, kips (N)
F_{in} = nominal bond stress = 60 psi (0.40 MPa)
D = diameter of HSS, in. (mm)

C_{in} = 1 if the filled composite column extends only above or below the point of load transfer
= 2 if the filled composite column extends both above and below the point of load transfer

As with encased columns, this specification assumes that the most advantageous combination of loaded area and concrete area are used to determine bearing strength. Thus, the nominal bearing strength is taken as $1.7 f_c' A_B$.

2f. Detailing Requirements

When shear connectors are used in filled composite columns, the provisions require that they be placed a distance of 2.5 times the width of a rectangular HSS or 2.5 times the diameter of a round HSS, both above and below the load transfer region. In most such situations, a uniform spacing is appropriate. However, when large forces are applied, other connector arrangements may be needed to avoid overloading the steel section or concrete core to which the load is applied directly.

I3. FLEXURAL MEMBERS

1. General

Three types of composite beams are addressed in this section: fully encased steel beams, concrete-filled HSS, and steel beams with mechanical anchorage to the slab.

When a composite beam is controlled by deflection, the design should limit the behavior of the beam to the elastic range under serviceability load combinations. Alternatively, the amplification effects of inelastic behavior should be considered when deflection is checked.

It is often not practical to make accurate stiffness calculations of composite flexural members. Comparisons to short-term deflection tests indicate that the effective moment of inertia, I_{eff}, is 15 to 30 percent lower than that calculated based on linear elastic theory (I_{equiv}). Therefore, for realistic deflection calculations, I_{eff} should be taken as 0.75 I_{equiv}.

As an alternative, one may use a lower bound moment of inertia, I_{lb}, as defined below:

$$I_{lb} = I_s + A_s(Y_{ENA} - d_3)^2 + (\Sigma Q_n / F_y)(2d_3 + d_1 - Y_{ENA})^2 \qquad \text{(C-I3-1)}$$

where

A_s = area of steel cross section, in.^{2} (mm^2)
d_1 = distance from the compression force in the concrete to the top of the steel section, in. (mm)
d_3 = distance from the resultant steel tension force for full section tension yield to the top of the steel, in. (mm)
I_{lb} = lower bound moment of inertia, in.^{4} (mm^4)

I_s = moment of inertia for the structural steel section, in.^{4} (mm^4)
ΣQ_n = sum of the nominal strengths of shear connectors between the point of maximum positive moment and the point of zero moment to either side, kips (kN)
Y_{ENA} = $[(A_s d_3 + (\Sigma Q_n/F_y)(2d_3 + d_1))/(A_s + (\Sigma Q_n/F_y))]$

The use of constant stiffness in elastic analyses of continuous beams is analogous to the practice in reinforced concrete design. The stiffness calculated using a weighted average of moments of inertia in the positive moment region and negative moment regions may take the following form:

$$I_t = aI_{pos} + bI_{neg} \tag{C-I3-2}$$

where

I_{pos} = effective moment of inertia for positive moment, in.^{4} (mm^4)
I_{neg} = effective moment of inertia for negative moment, in.^{4} (mm^4)

The effective moment of inertia is based on the cracked transformed section considering the degree of composite action. For continuous beams subjected to gravity loads only, the value of a may be taken as 0.6 and the value of b may be taken as 0.4. For composite beams used as part of a lateral force resisting system in moment frames, the value of a and b may be taken as 0.5 for calculations related to drift.

In cases where elastic behavior is desired, the cross-sectional strength of composite members is based on the superposition of elastic stresses including consideration of the effective section modulus at the time each increment of load is applied. For cases where elastic properties of partially composite beams are needed, the elastic moment of intertia may be approximated by

$$I_{eff} = I_s + \sqrt{(\Sigma Q_n/C_f)}\,(I_{tr} - I_s) \tag{C-I3-3}$$

where

I_s = moment of inertia for the structural steel section, in.^{4} (mm^4)
I_{tr} = moment of inertia for the fully composite uncracked transformed section, in.^{4} (mm^4)
ΣQ_n = strength of shear connectors between the point of maximum positive moment and the point of zero moment to either side, kips (N)
C_f = compression force in concrete slab for fully composite beam; smaller of $A_s F_y$ and $0.85 f'_c A_c$, kips (N)
A_c = area of concrete slab within the effective width, in.^{2} (mm^2)

The effective section modulus S_{eff}, referred to the tension flange of the steel section for a partially composite beam, may be approximated by

$$S_{eff} = S_s + \sqrt{(\Sigma Q_n/C_f)}\,(S_{tr} - S_s) \tag{C-I3-4}$$

where

S_s = section modulus for the structural steel section, referred to the tension flange, in.^{3} (mm^3)

S_{tr} = section modulus for the fully composite uncracked transformed section, referred to the tension flange of the steel section, in.^{3} (mm^3)

Equations C-I3-3 and C-I3-4 should not be used for ratios, $\Sigma Q_n / C_f$, less than 0.25. This restriction is to prevent excessive slip, as well as substantial loss in beam stiffness. Studies indicate that Equations C-I3-3 and C-I3-4 adequately reflect the reduction in beam stiffness and strength, respectively, when fewer connectors are used than required for full composite action (Grant and others, 1977).

U.S. practice does not generally require the following items be considered. They are highlighted here for a designer who chooses to construct something for which these items might apply.

1. Horizontal shear strength of the slab: For the case of girders with decks with narrow troughs or thin slabs, shear strength of the slab may govern the design (for example, see Figure C-I3.1). Although the configuration of decks built in the U.S. tends to preclude this mode of failure, it is important that it be checked if the force in the slab is large or an unconventional assembly is chosen. The shear strength of the slab may be calculated as the superposition of the shear strength of the concrete plus the contribution of any slab steel crossing the shear plane. The required shear strength, as shown in the figure, is given by the difference in the force between the regions inside and outside the potential failure surface. Where experience has shown that longitudinal cracking detrimental to serviceability is likely to occur, the slab should be reinforced in the direction transverse to the supporting steel section. It is recommended that the area of such reinforcement be at least 0.002 times the concrete area in the longitudinal direction of the beam and that it be uniformly distributed.

2. Rotational capacity of hinging zones: There is no required rotational capacity for hinging zones. Where plastic redistribution to collapse is allowed, the moments

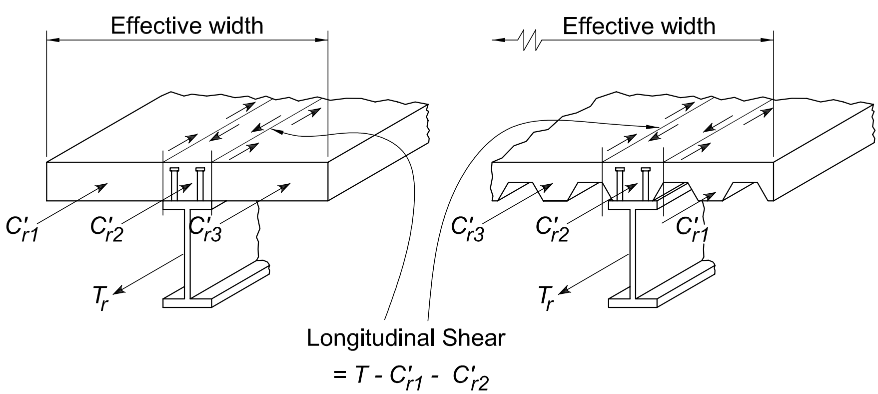


Fig. C-I3.1. Longitudinal shear in the slab (after Chien and Ritchie, 1984).

at a cross section may be as much as 30 percent lower than those given by a corresponding elastic analysis. This reduction in load effects is predicated, however, on the ability of the system to deform through very large rotations. To achieve these rotations, very strict local buckling and lateral-torsional buckling requirements must be fulfilled (Dekker and others, 1995). For cases in which a 10 percent redistribution is utilized (see Appendix 1), the required rotation capacity is within the limits provided by the local and lateral-torsional buckling provisions of Chapter F. Therefore, a rotational capacity check is not normally required for designs using this provision.

3. Minimum amount of shear connection: There is no minimum requirement for the amount of shear connection. Design aids in the U.S. often limit partial composite action to a minimum of 25 percent for practical reasons, but two issues arise with the use of low degrees of partial composite action. First, less than 50 percent composite action requires large rotations to reach the available flexural strength of the member and can result in very limited ductility after the nominal strength is reached. Second, low composite action results in an early departure from elastic behavior in both the beam and the studs. The current provisions, which are based on ultimate strength concepts, have eliminated checks for ensuring elastic behavior under service load combinations, and this can be an issue if low degrees of partial composite action are used.

4. Long-term deformations due to shrinkage and creep: There is no direct guidance in the computation of the long-term deformations of composite beams due to creep and shrinkage. The long-term deformation due to shrinkage can be calculated with the simplified model shown in Figure C-I3.2, in which the effect of shrinkage is taken as an equivalent set of end moments given by the shrinkage force (long-term restrained shrinkage strain times modulus of concrete times effective area of concrete) times the eccentricity between the center of the slab and the elastic neutral axis. If the restrained shrinkage coefficient for the aggregates is not known, the shrinkage strain for these calculations may be taken as 0.02 percent. The long-term deformations due to creep, which can be quantified using a model similar to that shown in the figure, are small unless the spans are long and the permanent live loads large. For shrinkage and creep effects, special attention should be given to lightweight aggregates, which tend to have higher creep coefficients and moisture absorption and lower modulus of elasticity than conventional aggregates, exacerbating any potential deflection problems. Engineering judgment is required, as calculations for long-term deformations require consideration of the many variables involved and because linear superposition of these effects is not strictly correct (ACI, 1997; Viest and others, 1997).

1a. Effective Width

The same effective width rules apply to composite beams with a slab on either one side or both sides of the beam. In cases where the effective stiffness of a beam with a one-sided slab is important, special care should be exercised since this model can substantially overestimate stiffness (Brosnan and Uang, 1995). To simplify design, effective width is based on the full span, center-to-center of supports, for both simple and continuous beams.

1b. Shear Strength

A conservative approach to shear provisions for composite beams is adopted by assigning all shear to the steel section web. This neglects any concrete slab contribution and serves to simplify design.

1c. Strength during Construction

Composite beam design requires care in considering the loading history. Loads applied to an unshored beam before the concrete has cured are resisted by the steel section alone, and only loads applied after the concrete has cured are considered to be resisted by the composite section. It is usually assumed for design purposes that concrete has hardened when it attains 75 percent of its design strength. Unshored beam deflection caused by fresh concrete tends to increase slab thickness and dead load. For longer spans this may lead to instability analogous to roof ponding. Excessive increase of slab thickness may be avoided by beam camber. Pouring the slab to a constant thickness will also help eliminate the possibility of ponding instability (Ruddy, 1986). When forms are not attached to the top flange, lateral bracing of the steel beam during construction may not be continuous and the unbraced length may control flexural strength, as defined in Chapter F.

This Specification does not include special requirements for strength during construction. For these noncomposite beams, the provisions of Chapter F apply.

Load combinations for construction loads should be determined for individual projects according to local conditions, using ASCE (2002) as a guide.

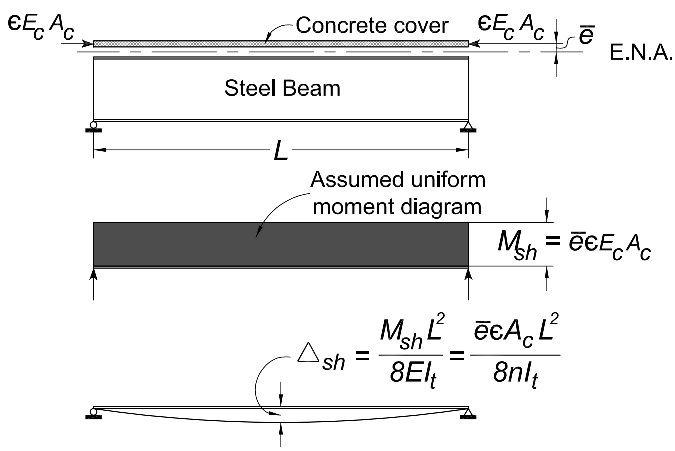


Fig. C-I3.2. Calculation of shrinkage effects [from Chien and Ritchie (1984)].

2. Strength of Composite Beams with Shear Connectors

Section I3.2 applies to simple and continuous composite beams with shear connectors, constructed with or without temporary shores.

2a. Positive Flexural Strength

The flexural strength of a composite beam in the positive moment region may be controlled by the strength of the steel section, the concrete slab or the shear connectors. In addition, web buckling may limit flexural strength if the web is slender and a large portion of the web is in compression.

According to Table B5.1, local web buckling does not reduce the plastic strength of a bare steel beam if the beam depth-to-web thickness ratio is not larger than $3.76\sqrt{E/F_y}$. In the absence of web buckling research on composite beams, the same ratio is conservatively applied to composite beams.

For beams with more slender webs, this Specification conservatively adopts first yield as the flexural strength limit. In this case, stresses on the steel section from permanent loads applied to unshored beams before the concrete has cured must be superimposed on stresses on the composite section from loads applied to the beams after hardening of concrete. For shored beams, all loads may be assumed to be resisted by the composite section.

When first yield is the flexural strength limit, the elastic transformed section is used to calculate stresses on the composite section. The modular ratio, $n = E/E_c$, used to determine the transformed section, depends on the specified unit weight and strength of concrete.

2b. Negative Flexural Strength

Loads applied to a continuous composite beam with shear connectors throughout its length, after the slab is cracked in the negative moment region, are resisted in that region by the steel section and by properly anchored longitudinal slab reinforcement. When an adequately braced compact steel section and adequately developed longitudinal reinforcing bars act compositely in the negative moment region, the nominal flexural strength is determined from plastic stress distributions.

2c. Strength of Composite Beams with Formed Steel Deck

Figure C-I3.3 is a graphic presentation of the terminology used in Section I3.2c.

The design rules for composite construction with formed steel deck are based upon a study (Grant and others, 1977) of the then-available test results. The limiting parameters listed in Section I3.2c were established to keep composite construction with formed steel deck within the available research data.

The minimum spacing of 18 in. for connecting composite decking to the support is intended to address a minimum uplift requirement during the construction phase prior to placing concrete.

2d. Shear Connectors

(1) Load Transfer for Positive Moment

When studs are used on beams with formed steel deck, they may be welded directly through the deck or through prepunched or cut-in-place holes in the deck. The usual procedure is to install studs by welding directly through the deck; however, when the deck thickness is greater than 16 gage (1.5 mm) for single thickness, or 18 gage (1.2 mm) for each sheet of double thickness, or when the total thickness of galvanized coating is greater than 1.25 ounces/sq. ft (0.38 kg/m^2), special precautions and procedures recommended by the stud manufacturer should be followed.

Composite beam tests in which the longitudinal spacing of shear connectors was varied according to the intensity of the static shear, and duplicate beams

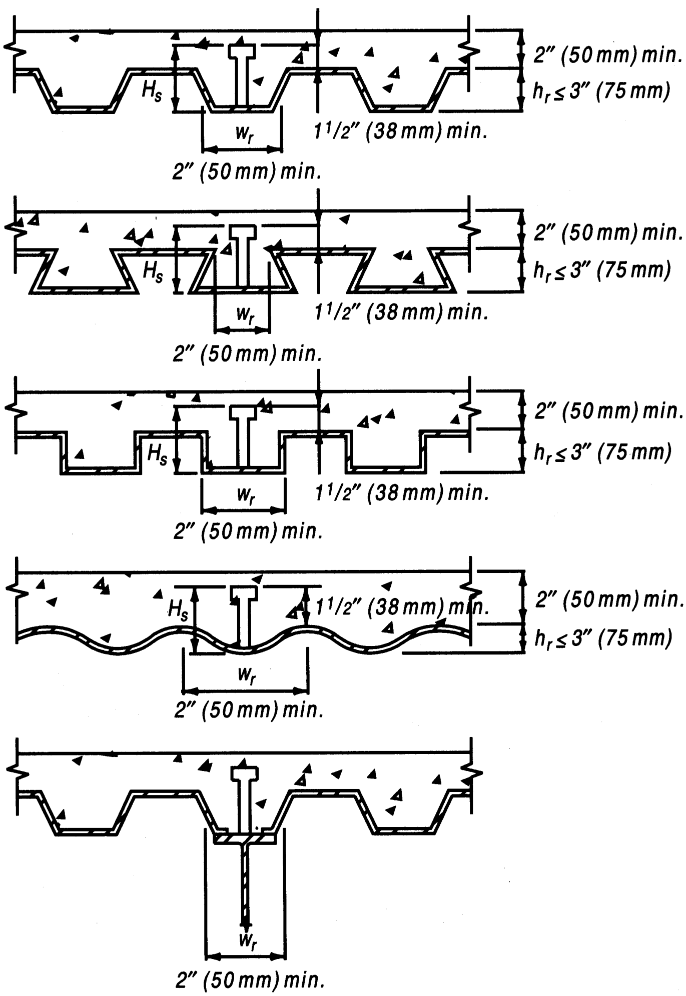


Fig. C-I3.3. Steel deck limits.

in which the connectors were uniformly spaced, exhibited approximately the same ultimate strength and approximately the same amount of deflection at nominal loads. Under distributed load conditions, only a slight deformation in the concrete near the more heavily stressed connectors is needed to redistribute the horizontal shear to other less heavily stressed connectors. The important consideration is that the total number of connectors be sufficient to develop the shear on either side of the point of maximum moment. The provisions of this Specification are based upon this concept of composite action.

In computing the available flexural strength at points of maximum negative bending, reinforcement parallel to the steel beam within the effective width of the slab may be included, provided such reinforcement is properly anchored beyond the region of negative moment. However, shear connectors are required to transfer the ultimate tensile force in the reinforcement from the slab to the steel beam.

When steel deck includes units for carrying electrical wiring, crossover headers are commonly installed over the cellular deck perpendicular to the ribs. These create trenches that completely or partially replace sections of the concrete slab above the deck. These trenches, running parallel to or transverse to a composite beam, may reduce the effectiveness of the concrete flange. Without special provisions to replace the concrete displaced by the trench, the trench should be considered as a complete structural discontinuity in the concrete flange.

When trenches are parallel to the composite beam, the effective flange width should be determined from the known position of the trench.

Trenches oriented transverse to composite beams should, if possible, be located in areas of low bending moment and the full required number of studs should be placed between the trench and the point of maximum positive moment. Where the trench cannot be located in an area of low moment, the beam should be designed as noncomposite.

(3) Strength of Stud Shear Connectors

Considerable research has been published in recent years indicating that shear stud strength equations in previous AISC Specifications are unconservative. Specifically, it has been understood for some time that the stud strength values given by Equation I5-1 in previous LRFD Specifications, in combination with the old Equations I3-2 and I3-3, which modified the strength based on whether the deck was perpendicular or parallel to the beams, are higher than those derived from either pushout or beam tests for studs embedded in modern steel decks (Jayas and Hosain, 1988; 1988a; Mottram and Johnson, 1990; Easterling, Gibbings, and Murray, 1993; Roddenberry and others, 2002) Equation I5-1 in the previous specifications is similar to the new Equation I3-5 but without the R_g and R_p factors.

Other codes use a stud strength expression similar to the previous AISC *LRFD Specification*; the stud strength is reduced by a ϕ factor of 0.8 in the Canadian code (CSA, 1994) and by an even lower partial safety factor ($\phi = 0.60$) for the corresponding stud strength equations in Eurocode 4 (2003).

The origin of this discrepancy can be traced to the way the old equations for stud strength were developed. The old approach was developed based on tests on solid slabs, and, as noted by the current R_p and R_g factors in the new Equation I3-4, the current approach remains valid for this case. Following studies reported in Robinson (1967) and Fisher (1970), Grant and others developed expressions for stud strength that accounted for the presence of the steel deck by including additional variables related to the deck and stud geometries (Grant and others, 1977). However, most of those tests were conducted with decks that were formed specifically for the tests from flat steel sheets.

The majority of composite steel floor decks used today have a stiffening rib in the middle of each deck flute. Because of the stiffener, studs must be welded off-center in the deck rib. Recent studies have shown that shear studs behave differently depending upon their location within the deck rib (Lawson, 1992; Easterling and others, 1993; Van der Sanden, 1995; Yuan, 1996; Johnson and Yuan, 1998; Roddenberry and others, 2002; Roddenberry and others, 2002a). The so-called "weak" (unfavorable) and "strong" (favorable) positions are illustrated in Figure C-I3.4. Furthermore, the maximum value shown in these studies for studs welded through steel deck is on the order of 0.7 to 0.75 $F_u A_{sc}$. Studs placed in the weak position have strengths as low as $0.5 F_u A_{sc}$.

The strength of stud connectors installed in the ribs of concrete slabs on formed steel deck with the ribs oriented perpendicular to the steel beam is reasonably estimated by the strength of stud connectors computed from Equation I3-3, which sets the default value for shear stud strength equal to that for the weak stud position. Both AISC (1997) and the Steel Deck Institute (SDI, 1999) recommend that studs be detailed in the strong position, but ensuring that studs are placed in the strong position is not necessarily an easy task because it is not always easy for the installer to determine where along the beam the particular rib is located, relative to the end, midspan or point of zero shear. Therefore, the installer may not be clear on which is the strong and which is the weak position.

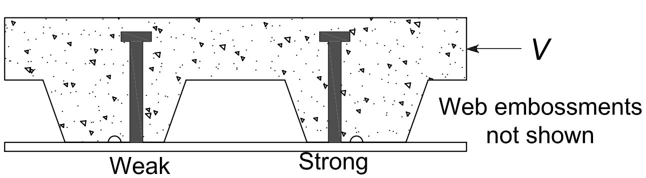


Fig. C-I3.4. Weak and strong stud positions [from Roddenberry and others (2002a)].

In most composite floors designed today, the ultimate strength of the composite section is governed by the stud strength, as full composite action is typically not the most economical solution to resist the required strength. The degree of composite action, as represented by the ratio $\Sigma Q_n/F_y A_s$ (the total shear connection strength divided by the yield strength of the steel cross section), influences the flexural strength as shown in Figure C-I3.5.

It can be seen from Figure C-I3.5 that a relatively large change in shear connection strength results in a much smaller change in flexural strength. Thus, formulating the influence of steel deck on shear connector strength by conducting beam tests and back-calculating through the flexural model, as was done in the past, lead to an inaccurate assessment of stud strength when installed in metal deck.

The changes in the 2005 Specification are not a result of either structural failures or performance problems. Designers concerned about the strength of existing structures need to note that the slope of the curve shown in Figure C-I3.5 is rather flat as the degree of composite action approaches one. Thus, even a large change in shear stud strength does not result in a proportional decrease of the flexural strength. In addition, as noted above, the current expression does not account for all the possible shear force transfer mechanisms, primarily because many of them are difficult or impossible to quantify. However, as noted in the Commentary to Section I3.1, as the degree of composite action decreases, the deformation demands on shear studs increase. This effect is reflected by the increasing slope of the relationship shown in Figure C-I3.5 as the degree of composite action decreases. Thus designers should be careful

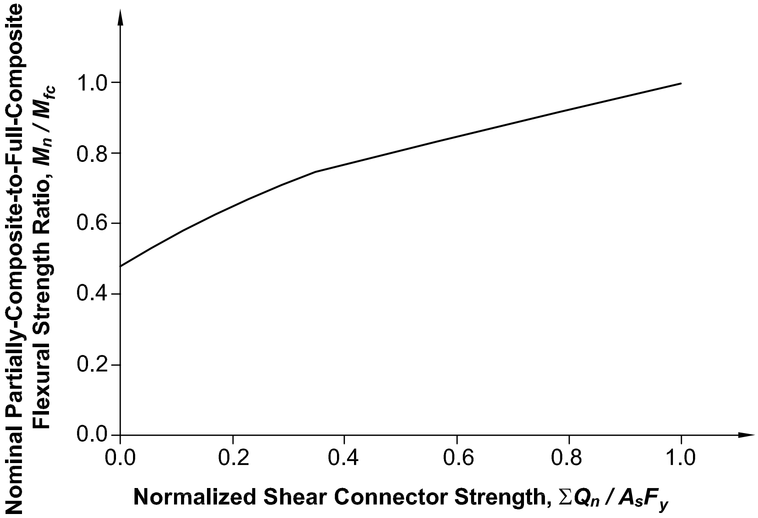


Fig. C-I3.5. Normalized flexural strength versus shear connection strength ratio [W16×31 (W410×46.1), $F_y = 50$ ksi (345 MPa), $Y2 = 4.5$ in. (114 mm)] (after Easterling and others, 1993).

when evaluating the strength of existing composite beams with 50 percent composite action or less.

(4) Strength of Channel Shear Connectors

Equation I3-4 is a modified form of the formula for the strength of channel connectors presented in Slutter and Driscoll (1965), which was based on the results of pushout tests and a few simply supported beam tests with solid slabs by Viest and others (1952). The modification has extended its use to lightweight concrete.

Eccentricities need not be considered in the weld design for cases where the welds at the toe and heel of the channel are greater than 3/16 in. and the connector meets the following requirements:

$$1.0 \le \frac{t_f}{t_w} \le 5.5$$

$$\frac{H}{t_w} \ge 8.0$$

$$\frac{L_c}{t_f} \ge 6.0$$

$$0.5 \le \frac{R}{t_w} \le 1.6$$

where t_f is the connector flange thickness, t_w is the connector web thickness, H is the height of the connector, L_c is the length of the connector, and R is the radius of the fillet between the flange and web of the connector.

(6) Shear Connector Placement and Spacing

Uniform spacing of shear connectors is permitted, except in the presence of heavy concentrated loads.

Studs not located directly over the web of a beam tend to tear out of a thin flange before attaining full shear-resisting strength. To guard against this contingency, the size of a stud not located over the beam web is limited to 2 1/2 times the flange thickness (Goble, 1968). The practical application of this limitation is to select only beams with flanges thicker than the stud diameter divided by 2.5.

The minimum spacing of connectors along the length of the beam, in both flat soffit concrete slabs and in formed steel deck with ribs parallel to the beam, is six diameters; this spacing reflects development of shear planes in the concrete slab (Ollgaard and others, 1971). Because most test data are based on the minimum transverse spacing of four diameters, this transverse spacing was set as the minimum permitted. If the steel beam flange is narrow, this spacing requirement may be achieved by staggering the studs with a minimum transverse spacing of three diameters between the staggered row of studs. When deck ribs are parallel to the beam and the design requires more studs than can be placed in the rib, the deck may be split so that adequate spacing is available for stud installation. Figure C-I3.6 shows possible connector arrangements.

3. Flexural Strength of Concrete-Encased and Filled Members

Tests of concrete-encased beams demonstrated that: (1) the encasement drastically reduces the possibility of lateral-torsional instability and prevents local buckling of the encased steel; (2) the restrictions imposed on the encasement practically prevent bond failure prior to first yielding of the steel section; and (3) bond failure does not necessarily limit the moment strength of an encased steel beam (ASCE, 1979). Accordingly, this Specification permits three alternative design methods for determination of the nominal flexural strength: (a) based on the first yield in the tension flange of the composite section; (b) based on the plastic flexural strength of the steel section alone; and (c) based on the plastic flexural strength of the composite section or the strain-compatibility method. Method (c) is applicable only when shear connectors are provided along the steel section and reinforcement of the concrete encasement meets the specified detailing requirements. No limitations are placed on the slenderness of either the composite beam or the elements of the steel section, since the encasement effectively inhibits both local and lateral buckling.

In method (a), stresses on the steel section from permanent loads applied to unshored beams before the concrete has hardened must be superimposed on stresses on the composite section from loads applied to the beams after hardening of the concrete. In this superposition, all permanent loads should be multiplied by the dead load factor and all live loads should be multiplied by the live load factor. For shored beams, all loads may be assumed as resisted by the composite section. Complete interaction (no slip) between the concrete and steel is assumed.

I4. COMBINED AXIAL FORCE AND FLEXURE

As with all frame analyses in this Specification, required strengths for composite beam-columns should be obtained from second-order analysis or amplified first-order analysis. With respect to the assessment of the available strength, the Specification provisions for interaction between axial force and flexure in composite

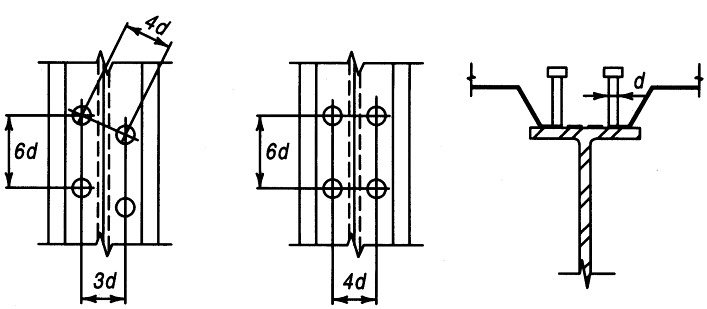


Fig. C-I3.6. Shear connector arrangements.

members do not provide explicit equations. However, the provisions provide guidance in Section I4 on the principles that can serve to establish an interaction diagram similar to those used in reinforced concrete design.

Composite Beam-Columns. The available axial strength, including the effects of buckling, and the available flexural strength can be calculated using either the plastic stress distribution method or the strain-compatibility method. There are three simplified approaches for determining the strength of composite beam-columns discussed below that take advantage of strength determination for a limited number of cases and interpolation for all other cases.

The first approach applies to doubly symmetric composite beam-columns, the most common geometry found in building construction. For this case, the interaction equations of Section H1 provide a conservative assessment of the available strength of the member for combined axial compression and flexure. These provisions may also be used for combined axial tension and flexure. The degree of conservatism generally depends on the extent of concrete contribution to the overall strength, relative to the steel contribution. Thus, for example, the equations are generally more conservative for members with high concrete compressive strength as compared to members with low concrete compressive strength.

The second approach is based on developing interaction surfaces for combined axial compression and flexure at the nominal strength level, using the plastic stress distribution method. This results in interaction surfaces similar to those shown in Figures C-I1.1 and C-I4.1. The five points identified in Figure C-I4.1 are defined by the plastic stress distribution used in their determination. Point A is the pure axial strength determined according to Equations I2-2 or I2-13. Point B is determined as the flexural strength of the section determined according to the provisions of Section I3. Point C corresponds to a plastic neutral axis location that results in the same flexural capacity as Point B but with axial load. Point D corresponds to an axial strength of one half of that determined for Point C. Point E is an arbitrary point necessary to better reflect bending strength for y-axis bending of encased shapes and bending of filled HSS. Linear interpolation between these anchor points may be used. However, with this approach, care should be taken in reducing Point D by a resistance factor or to account for member slenderness, as that may lead to an unsafe situation whereby additional flexural strength is permitted at a lower axial compressive strength than predicted by the cross section strength of the member. Once the nominal strength interaction surface is determined, length effects according to Equations I2-6 and I2-7 must be applied. The available strength is then determined by applying the compression and bending resistance factors or safety factors.

The third approach is a simplified bilinear approach as shown in Figure C-I4.1. After the column axial strength (Point A in Figure C-I4.1) is computed using Equation I2-2 for concrete-encased sections or Equation I2-13 for concrete-filled sections, this strength is reduced by the length effects using Equations E2-6 or E2-7 to obtain P_n, or Point A_λ. The resistance factor, ϕ_c, or safety factor, Ω_c, is then applied to this value to become the anchor point for design on the vertical axis, A_d. The anchor point on the horizontal axis, Point B_d, is given by the flexural strength of the section, Point B, modified by the appropriate bending resistance factor or safety factor.

Point C is then adjusted downward by the same length effect reduction as applied to Point A, to obtain Point C_λ. Point C_λ is then adjusted down by ϕ_c or Ω_c and to the left by ϕ_b or Ω_b to obtain Point C_d. A straight line approximation may then be used between Points A_d, C_d and B_d, as shown in the figure. Using linear interpolation between Points A_d, C_d and B_d in Figure C-I4.1, the following interaction equations may be derived for composite beam-columns subjected to combined axial compression plus biaxial flexure:

If $P_r < P_C$

$$\frac{M_{rx}}{M_{Cx}} + \frac{M_{ry}}{M_{Cy}} \leq 1 \qquad \text{(C-I4-1a)}$$

If $P_r \geq P_C$

$$\frac{P_r - P_C}{P_A - P_C} + \frac{M_{rx}}{M_{Cx}} + \frac{M_{ry}}{M_{Cy}} \leq 1 \qquad \text{(C-I4-1b)}$$

where

P_r = required compressive strength, kips (N)
P_A = available axial compressive strength at Point A, kips (N)
P_C = available axial compressive strength at Point C, kips (N)
M_r = required flexural strength, kip-in. (N-mm)
M_C = available flexural strength at Point C, kip-in. (N-mm)
x = subscript relating symbol to strong axis bending
y = subscript relating symbol to weak axis bending

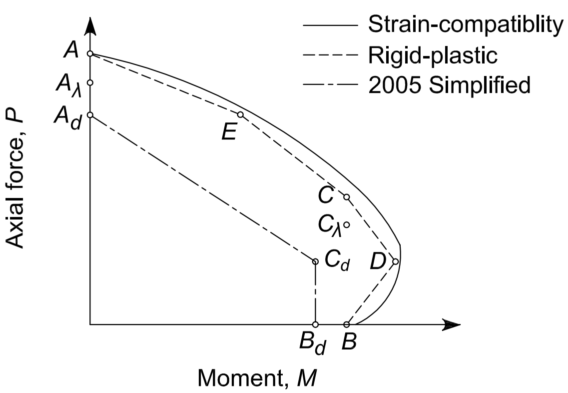


Fig. C-I4.1. Interaction diagram for composite beam-column design.

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

P_r $= P_u =$ required compressive strength using LRFD load combinations, kips (N)
P_A $= P_{Ad} =$ design axial compressive strength, determined in accordance with Section I2, kips (N)
P_C $= P_{Cd} =$ design axial compressive strength at point C_d in Figure C-I4.1, kips (N)
M_r = required flexural strength using LRFD load combinations, kip-in. (N-mm)
$M_C = M_{Cd} = M_{Bd}$ design flexural strength, determined in accordance with Section I3, kip-in. (N-mm)
ϕ_c $= 0.75$
ϕ_b $= 0.90$

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

P_r $= P_a =$ required compressive strength using ASD load combinations, kips (N)
P_A $= P_{Ad} =$ allowable compressive strength, determined in accordance with Section I2, kips (N)
P_C $= P_{Cd} =$ allowable axial compressive strength at Point C_d in Figure C-I4.1, kips (N)
M_r = required flexural strength using ASD load combinations, kip-in. (N-mm)
$M_C = M_{Cd} = M_{Bd}$ allowable flexural strength, determined in accordance with Section I3, kip-in. (N-mm).
Ω_c $= 2.00$
Ω_b $= 1.67$

For biaxial bending, the value of P_C may be different when computed for the major and minor axis. The smaller of the two values should be used in Equation C-I4-1b and for the limits of Equations C-I4-1a and b.

Composite Beams Subjected to Combined Axial Force and Flexure. Combined axial force and flexure on composite floor beams has not been addressed directly in this chapter. Composite floor beam members (steel beams composite with floor slabs at their top flange) with axial loading may include collector elements (drag struts) and stabilizing elements for sloping column members. Few detailed design guidelines exist for such members; preliminary guidance for seismic design is given in AISC (2002).

Load combinations as set forth in ASCE (2002) should be used to determine the applicable loading at critical sections, including secondary bending effects of eccentrically applied loading. Adequate means to transmit axial loading to and from the steel section should be provided. Where shear connectors are used, the top flanges may be considered braced for compressive loading at the shear connector locations. Additional shear connectors may be required for axial load transfer and added flexure as determined from the load combinations. For shear studs added to transfer axial loads between beams and slabs, Q_n may be determined in accordance with Section I3. For load combinations resulting in compressive loading of the lower flange, length effects between brace points should be considered. Inflection points should not be considered brace points for torsional buckling of the unbraced flange. For discussion and design methodology, the reader is referred to Galambos (1998).

I5. SPECIAL CASES

Tests are required for construction that falls outside the limits given in this Specification. Different types of shear connectors may require different spacing and other detailing than stud and channel connectors.

CHAPTER J

DESIGN OF CONNECTIONS

The provisions of Chapter J cover the design of connections not subject to *cyclic loads*. Wind and other environmental loads are generally not considered to be *cyclic loads*. The provisions generally apply to connections other than HSS and box members. See Chapter K for HSS and box member connections and Appendix 3 for fatigue provisions.

J1. GENERAL PROVISIONS

Selection of weld type (CJP versus fillet versus PJP) depends on base connection geometry (butt versus T or corner) and required strength, among other issues discussed in this Section. Consideration of notch effects and the ability to evaluate with NDE may be appropriate for cyclically loaded joints or joints expected to deform plastically.

1. Design Basis

In the absence of defined design loads, a minimum design load should be considered. Historically, a value of 10 kips (44 kN) for LRFD and 6 kips (27 kN) for ASD have been used as reasonable values. For elements such as lacing, sag rods, girts or small simple members, a load more appropriate to the size and use of the part should be used. Design requirements for the installed elements as well as construction loads need to be considered when specifying minimum loads for connections.

2. Simple Connections

Simple connections are considered in Sections B3.6a. and J1.2. In Section B3.6a "simple" connections are defined (with further elaboration in the Commentary for Section B3.6) as a guide to idealization of the structure for the purpose of analysis. The assumptions made in the analysis determine the outcome of the analysis that serves as the basis for design (for connections that means the force and deformation demands that the connection must resist). Section J1.2 focuses on the actual proportioning of the connection elements to achieve the required resistance. In short, Section B3.6a establishes the modeling assumptions that determine the design forces and deformations for use in Section J1.2.

Sections B3.6a and J1.2 are not mutually exclusive. If a "simple" connection is assumed for analysis, the actual connection, as finally designed, must deliver performance consistent with that assumption. For a simple connection it is important to verify that the performance is consistent with the design assumptions; in other words, the connection must be able to meet the required rotation and must not introduce strength and stiffness that significantly alter the mode of response.

3. Moment Connections

Two types of moment connections are defined in Section B3.6b: fully restrained (FR) and partially restrained (PR). FR moment connections must have sufficient strength and stiffness to transfer moment and maintain the angle between connected members. PR moment connections are designed to transfer moments but also allow rotation between connected members as the loads are resisted. The response characteristics of a PR connection must be documented in the technical literature or established by analytical or experimental means. The component elements of a PR connection must have sufficient strength, stiffness and deformation capacity to satisfy the design assumptions.

4. Compression Members with Bearing Joints

The provisions for "compression members other than columns finished to bear" are intended to account for member out-of-straightness and also to provide a degree of robustness in the structure so as to resist unintended or accidental lateral loadings that may not have been considered explicitly in the design.

A provision analogous to that in Section J1.4(b)(i), requiring that splice materials and connectors have an available strength of at least 50 percent of the required compressive strength, has been in the AISC Specifications for more than 40 years. The current Specification clarifies this requirement by stating that the force for proportioning the splice materials and connectors is a tensile force. This avoids uncertainty as to how to handle situations where compression on the connection imposes no force on the connectors.

Proportioning the splice materials and connectors for 50 percent of the required member strength is simple, but can be very conservative. In Section J1.4(b)(ii), the Specification offers an alternative that addresses directly the design intent of these provisions. The lateral load of 2 percent of the required compressive strength of the member simulates the effect of a kink at the splice, caused by an end finished slightly out-of-square or other construction condition. Proportioning the connection for the resulting moment and shear also provides a degree of robustness in the structure.

5. Splices in Heavy Sections

Solidified but still hot filler metal contracts significantly as it cools to ambient temperature. Shrinkage of large groove welds between elements that are not free to move so as to accommodate the shrinkage causes strains in the material adjacent to the weld that can exceed the yield point strain. In thick material the weld shrinkage is restrained in the thickness direction, as well as in the width and length directions, causing triaxial stresses to develop that may inhibit the ability to deform in a ductile manner. Under these conditions, the possibility of *brittle fracture* increases.

When splicing hot-rolled shapes with flange thicknesses exceeding 2 in. (50 mm) or heavy welded built-up members, these potentially harmful weld shrinkage strains can be avoided by using bolted splices, fillet-welded lap splices, or splices that combine a welded and bolted detail (see Figure C-J1.1). Details and techniques that perform well for materials of modest thickness usually must be changed or supplemented by more demanding requirements when welding thick material. The provisions of AWS D1.1 (AWS, 2004) are minimum requirements that apply to most structural welding situations; however, when designing and fabricating welded splices of hot-rolled shapes with flange thicknesses exceeding 2 in. (50 mm) and similar built-up cross sections, special consideration must be given to all aspects of the welded splice detail:

(1) Notch-toughness requirements should be specified for tension members; see Commentary Section A3.
(2) Generously sized weld access holes (see Section J1.6) are required to provide increased relief from concentrated weld shrinkage strains, to avoid close juncture of welds in orthogonal directions, and to provide adequate clearance for the exercise of high quality workmanship in hole preparation, welding and for ease of inspection.
(3) Preheating for thermal cutting is required to minimize the formation of a hard surface layer.
(4) Grinding of copes and access holes to bright metal to remove the hard surface layer is required, along with inspection using magnetic particle or dye-penetrant methods, to verify that transitions are free of notches or cracks.

In addition to tension splices of truss chord members and tension flanges of flexural members, other joints fabricated from heavy sections subject to tension should be given special consideration during design and fabrication.

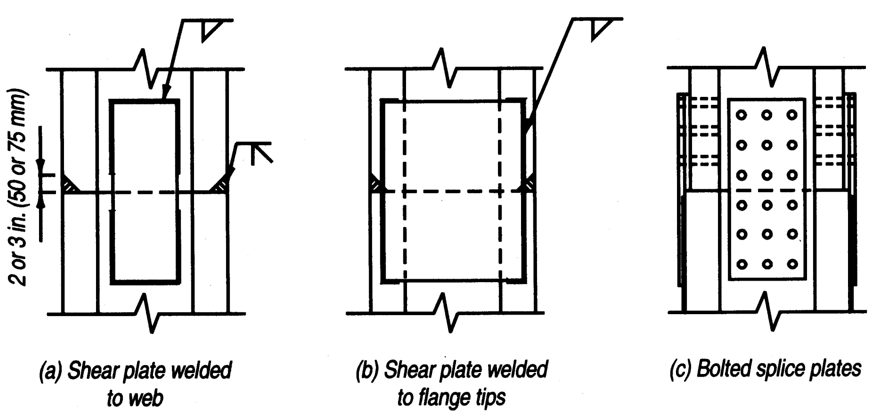


Fig. C-J1.1. Alternative splices that minimize weld restraint tensile stresses.

Previous editions of this Specification mandated that backing bars and weld tabs be removed from all splices of heavy sections. These requirements were deliberately removed from this edition, being judged unnecessary and, in some situations, potentially resulting in more harm than good. The Specification still permits the engineer of record to specify their removal when this is judged appropriate.

The previous requirement for the removal of backing bars necessitated, in some situations, that such operations be performed out-of-position; that is, the welding required to restore the backgouged area had to be applied in the overhead position. This may necessitate alternate equipment for gaining access, different welding equipment, processes and/or procedures, and other practical constraints. When box sections made of plate are spliced, access to the interior side (necessary for backing removal) is typically impossible.

Weld tabs that are left in place on splices act as “short attachments” and attract little stress. Even though it is acknowledged that weld tabs might contain regions of inferior quality weld metal, the stress concentration effect is minimized since little stress is conducted through the attachment.

6. Beam Copes and Weld Access Holes

Beam copes and weld access holes are frequently required in the fabrication of structural components. The geometry of these structural details can affect the components’ performance. The size and shape of beam copes and weld access holes can have a significant effect on the ease of depositing sound weld metal, the ability to conduct nondestructive examinations, and the magnitude of the stresses at the geometric discontinuities produced by these details.

Weld access holes used to facilitate welding operations are required to have a minimum length from the toe of the weld preparation (see Figure C-J1.2) equal to 1.5 times the thickness of the material in which the hole is made. This minimum length is expected to accommodate and relieve a significant amount of the weld shrinkage strains at the web-to-flange intersection.

The height of the weld access hole must provide sufficient clearance for ease of welding and inspection and must be large enough to allow the welder to deposit sound weld metal through and beyond the web. A weld access hole height equal to 1.5 times the thickness of the material with the access hole but not less than 1 in. (25 mm) has been judged to satisfy these welding and inspection requirements. The height of the weld access hole need not exceed 2 in. (50 mm).

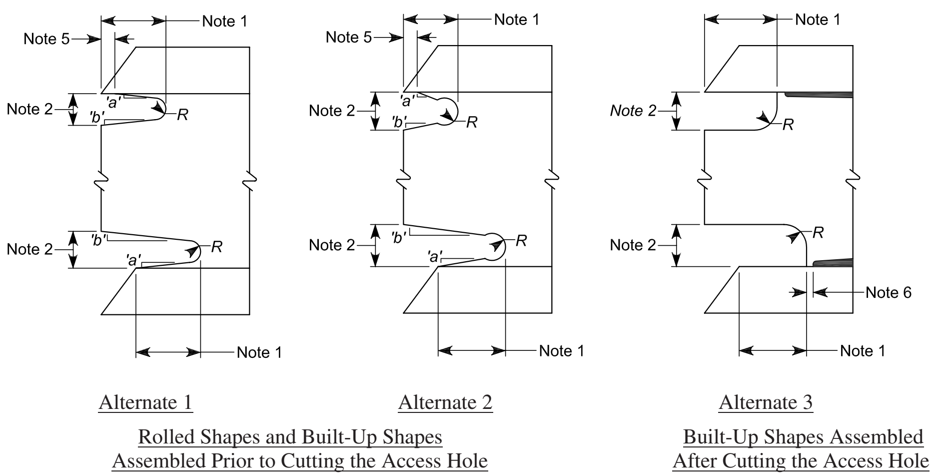
The geometry of the reentrant corner between the web and the flange determines the level of stress concentration at that location. A 90° reentrant corner having a very small radius produces a very high stress concentration that may lead to rupture of the flange. Consequently, to minimize the stress concentration at this location, the edge of the web shall be sloped or curved from the surface of the flange to the reentrant surface of the access hole.

Stress concentrations along the perimeter of beam copes and weld access holes also can affect the performance of the joint. Consequently, all beam copes and weld access holes are required to be free of stress raisers such as notches and gouges.

Stress concentrations at web-to-flange intersections of built-up shapes can be decreased by terminating the weld away from the access hole. Thus, for built-up shapes with fillet welds or partial-joint-penetration groove welds that join the web to the flange, the weld access hole may terminate perpendicular to the flange, provided that the weld is terminated at least one thickness of the web away from the access hole.

7. Placement of Welds and Bolts

Slight eccentricities between the gravity axis of single and double angle members and the center of gravity of connecting bolts or rivets have long been ignored as having negligible effect on the static strength of such members. Tests have shown that similar practice is warranted in the case of welded members in statically loaded structures (Gibson and Wake, 1942).



Notes: There are typical details for joints welded from one side against steel backing. Alternative details are discussed in the commentary text.

(1) Width: Greater of 1.5 × t_w or $1^1/_2$ in. (38 mm). Tolerance is ± $^1/_4$ in. (6 mm).

(2) Height: Greater of 1.5 t_w or 1 in. (25 mm) but need not exceed 2 in. (50 mm).

(3) R: $^3/_8$ in. min. (8 mm). Grind the thermally cut surfaces of access holes in heavy shapes as defined in Section A3.1c and A3.1d.

(4) Slope 'a' forms a transition from the web to the flange. Slope 'b' may be horizontal.

(5) The bottom of the top flange is to be contoured to permit the tight fit of backing bars where they are to be used.

(6) The web-to-flange weld of built-up members is to be held back a distance of at least the weld size from the edge of the access hole.

Fig. C-J1.2. Weld access hole geometry.

However, the fatigue life of eccentrically loaded welded angles has been shown to be very short (Kloppel and Seeger, 1964). Notches at the roots of fillet welds are harmful when alternating tensile stresses are normal to the axis of the weld, as could occur due to bending when axial cyclic loading is applied to angles with end welds not balanced about the neutral axis. Accordingly, balanced welds are required when such members are subjected to cyclic loading (see Figure C-J1.3).

8. Bolts in Combination with Welds

As in previous editions, this Specification does not permit bolts to share the load with welds except for bolts in shear connections. The conditions for load sharing have, however, changed substantially based on recent research (Kulak and Grondin, 2001). For shear-resisting connections with longitudinally loaded fillet welds, load sharing between the longitudinal welds and bolts in standard holes or short-slotted holes transverse to the direction of the load is permitted, but the contribution of the bolts is limited to 50 percent of the available strength of the equivalent bearing-type connection. Both A307 and high-strength bolts are permitted. The heat of welding near bolts will not alter the mechanical properties of the bolts.

In making alterations to existing structures, the use of welding to resist loads other than those produced by existing dead load present at the time of making the alteration is permitted for riveted connections and high-strength bolted connections if the bolts are pretensioned to the levels in Table J3.1 or J3.1M prior to welding.

The restrictions on bolts in combination with welds do not apply to typical bolted/welded beam-to-girder and beam-to-column connections and other comparable connections (Kulak, Fisher, and Struik, 1987).

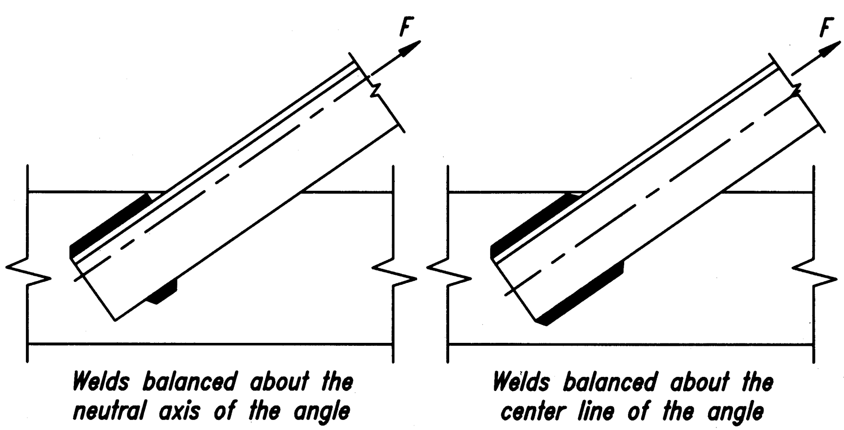


Fig. C-J1.3. Balanced welds.

9. High-Strength Bolts in Combination with Rivets

When high-strength bolts are used in combination with rivets, the ductility of the rivets permits the direct addition of the strengths of the two fastener types.

10. Limitations on Bolted and Welded Connections

Pretensioned bolts, slip-critical bolted connections, or welds are required whenever connection slip can be detrimental to the performance of the structure or there is a possibility that nuts will back off. Snug-tightened high-strength bolts are recommended for all other connections.

J2. WELDS

Selection of weld type [complete-joint-penetration (CJP) groove weld versus fillet versus partial-joint-penetration (PJP) groove weld] depends on base connection geometry (butt versus T or corner), in addition to required strength, and other issues discussed below. Consideration of notch effects and the ability to evaluate with nondestructive testing may be appropriate for cyclically loaded joints or joints expected to deform plastically.

1. Groove Welds

1a. Effective Area

Effective throats larger than those in Table J2.1 can be qualified by tests. The weld reinforcement is not used in determining the effective throat of a groove weld.

1b. Limitations

Table J2.3 provides a minimum size of PJP groove weld for a given thickness of the thinner part joined. Structural steel with a specified minimum yield stress of 50 ksi (350 MPa) is the prevalent material. The use of prequalified weld procedures is prevalent in structural welding. The minimum weld sizes required in this Specification are appropriate for filler metal prequalified with 50-ksi (350 MPa) base metal. Also, see the commentary to Section J2.2b for fillet weld limitations.

2. Fillet Welds

2a. Effective Area

The effective throat of a fillet weld is based on the root of the joint and the face of the diagrammatic weld; hence this definition gives no credit for weld penetration or reinforcement at the weld face. Some welding procedures produce a consistent penetration beyond the root of the weld. This penetration contributes to the strength of the weld. However, it is necessary to demonstrate that the weld procedure to be used produces this increased penetration. In practice, this can be done initially by cross-sectioning the runoff plates of the joint. Once this is done, no further testing is required, as long as the welding procedure is not changed.

2b. Limitations

Table J2.4 provides the minimum size of a fillet weld for a given thickness of the thinner part joined. The requirements are not based on strength considerations, but on the quench effect of thick material on small welds. Very rapid cooling of weld metal may result in a loss of ductility. Furthermore, the restraint to weld metal shrinkage provided by thick material may result in weld cracking. The use of the thinner part to determine the minimum size weld is based on the prevalence of the use of filler metal considered to be "low hydrogen." Because a $^5/_{16}$-in. (8 mm) fillet weld is the largest that can be deposited in a single pass by the SMAW process and still be considered prequalified under AWS D1.1, $^5/_{16}$ in. (8 mm) applies to all material $^3/_4$ in. (19 mm) and greater in thickness, but minimum preheat and interpass temperatures are required by AWS D1.1. Both the engineer of record and the shop welder must be governed by the requirements.

Table J2.3 gives the minimum effective throat thickness of a PJP groove weld. Notice that for PJP groove welds Table J2.3 goes up to a plate thickness of over 6 in. (150 mm) and a minimum weld throat of $^5/_8$ in. (16 mm), whereas for fillet welds Table J2.4 goes up to a plate thickness of over $^3/_4$ in. (19 mm) and a minimum leg size of fillet weld of only $^5/_{16}$ in. (8 mm). The additional thickness for PJP groove welds is intended to provide for reasonable proportionality between weld and material thickness.

For thicker members in lap joints, it is possible for the welder to melt away the upper corner, resulting in a weld that appears to be full size but actually lacks the required weld throat dimension. See Figure C-J2.1(a). On thinner members, the full weld throat is likely to be achieved, even if the edge is melted away.

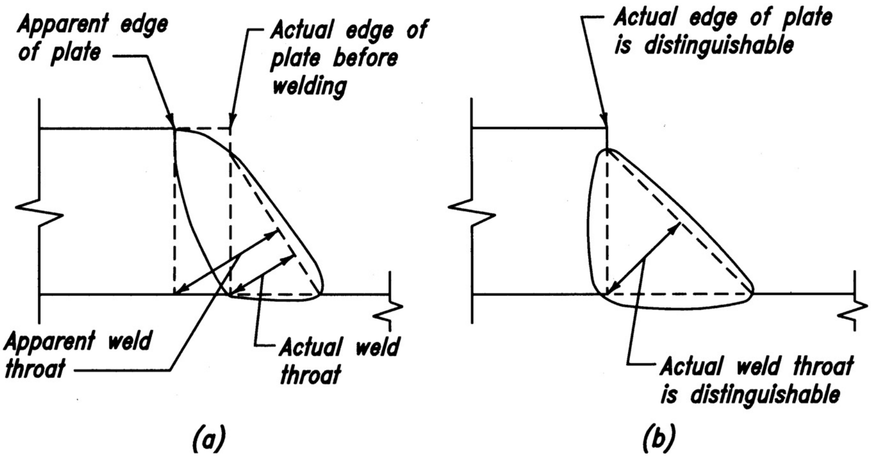


Fig. C-J2.1. Identification of plate edge.

Accordingly, when the plate is 1/4 in. (6 mm) or thicker, the maximum fillet weld size is 1/16 in. (2 mm) less than the plate thickness, ensuring that the edge remains behind [see Figure C-J2.1(b)].

Where longitudinal fillet welds are used alone in a connection (see Figure C-J2.2), Section J2.2b requires that the length of each weld be at least equal to the width of the connecting material because of shear lag (Freeman, 1930).

By providing a minimum lap of five times the thickness of the thinner part of a lap joint, the resulting rotation of the joint when pulled will not be excessive, as shown in Figure C-J2.3. Fillet welded lap joints under tension tend to open and apply a tearing action at the root of the weld as shown in Figure C-J2.4(b), unless restrained by a force F as shown in Figure C-J2.4(a).

End returns are not essential for developing the capacity of fillet welded connections and have a negligible effect on their strength. Their use has been encouraged to ensure that the weld size is maintained over the length of the weld, to enhance the fatigue resistance of cyclically loaded flexible end connections, and to increase the plastic deformation capability of such connections.

The weld capacity database on which the specifications were developed had no end returns. This includes the study reported in Higgins and Preece (1968), the

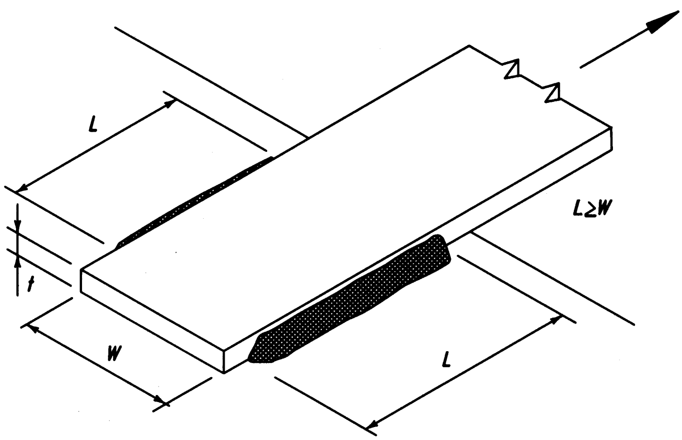


Fig. C-J2.2. Longitudinal fillet welds.

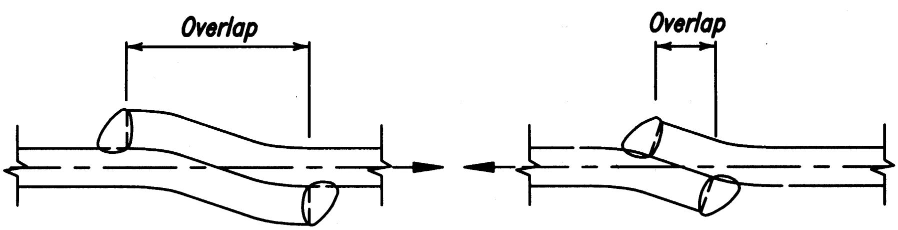


Fig. C-J2.3. Minimum lap.

seat angle tests in Lyse and Schreiner (1935), the seat and top angle tests in Lyse and Gibson (1937), the tests on beam webs welded directly to a column or girder by fillet welds in Johnston and Deits (1942), and the tests on eccentrically loaded welded connections reported in Butler, Pal, and Kulak (1972). Hence, the current strength values and joint-capacity models do not require end returns when the required weld size is provided. Johnston and Green (1940) noted that movement consistent with the design assumption of no end restraint (in other words, joint flexibility) was enhanced without end returns. They also verified that greater plastic deformation of the connection was achieved when end returns existed, although the strength was not significantly different.

When longitudinal fillet welds parallel to the stress are used to transmit the load to the end of an axially loaded member, the welds are termed "end loaded." Typical examples of such welds include, but are not limited to: (a) longitudinally welded lap joints at the end of axially loaded members; (b) welds attaching bearing stiffeners; and (c) similar cases. Typical examples of longitudinally loaded fillet welds are not considered end loaded include, but are not limited to: (a) welds that connect plates or shapes to form built-up cross sections in which the shear force is applied to each increment of length of weld depending upon the distribution of the shear along the length of the member; and (b) welds attaching beam web connection angles and shear plates because the flow of shear force from the beam or girder web to the weld is essentially uniform throughout the weld length; that is, the weld is not end-loaded despite the fact that it is loaded parallel to the weld axis. Neither does the reduction coefficient, β, apply to welds attaching stiffeners to webs because the stiffeners and welds are not subject to calculated axial stress but merely serve to keep the web flat.

The distribution of stress along the length of end-loaded fillet welds is not uniform and is dependent upon complex relationships between the stiffness of the longitudinal fillet weld relative to the stiffness of the connected materials. Experience has shown that when the length of the weld is equal to approximately 100 times the weld size or less, it is reasonable to assume the effective length is equal to or less than the actual length. For weld lengths greater than 100 times the weld size, the effective length should be taken less than the actual length. The reduction coefficient, β, provided in Section J2.2b is the equivalent to that given in Eurocode 3 (1992), which is a simplified approximation of exponential

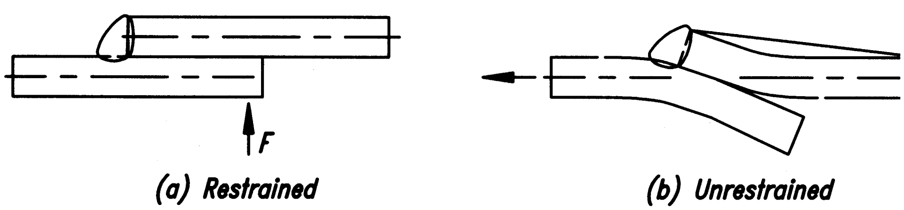


Fig. C-J2.4. Restraint of lap joints.

formulas developed by finite element studies and tests preformed in Europe over many years. The provision is based on the combined consideration of the nominal strength for fillet welds with leg size less than 1/4 in. (6 mm) and of a judgment-based serviceability limit of slightly less than 1/32 in. (1 mm) displacement at the end of the weld for welds with leg size 1/4 in. (6 mm) and larger. The mathematical form of the β factor implies that the minimum strength of an end-loaded weld is achieved when the length is approximately 300 times the leg size. Because it is illogical to conclude that the total strength of a weld longer than 300 times the weld size is more than that of a shorter weld, the length reduction coefficient is taken as 0.6 when the weld length is greater than 300 times the leg size.

In most cases, fillet weld terminations do not affect the strength or serviceability of connections. However, in certain cases the disposition of welds affect the planned function of the connection, and notches may affect the static strength and/or the resistance to crack initiation if *cyclic loads* of sufficient magnitude and frequency occur. For these cases, terminations before the end of the joint are specified to provide the desired profile and performance. In cases where profile and notches are less critical, terminations are permitted to be run to the end. In most cases, stopping the weld short of the end of the joint will not reduce the strength of the weld. The small loss of weld area due to stopping the weld short of the end of the joint by one to two weld sizes is not typically considered in the calculation of weld strength. Only short weld lengths will be significantly affected by this.

The following situations require special attention:

(1) For lapped joints where one part extends beyond the end or edge of the part to which it is welded and if the parts are subject to calculated tensile stress at the start of the overlap, it is important that the weld terminate a short distance from the stressed edge. For one typical example, the lap joint between the tee chord and the web members of a truss, the weld should not extend to the edge of the tee stem (see Figure C-J2.5). The best technique to avoid inadvertent notches at this critical location is to strike the welding arc at a point slightly back from the edge and proceed with welding in the direction away from the edge (see Figure C-J2.6). Where framing angles extend beyond the end of the beam web to which they are welded, the free end of the beam web is subject to zero stress; thus, it is permissible for the fillet weld to extend continuously across the top end, along the side and along the bottom end of the angle to the extreme end of the beam (see Figure C-J2.7).

(2) For connections such as framing angles and framing tees, which are assumed in the design of the structure to be *flexible connections*, the top and bottom edges of the outstanding legs or flanges must be left unwelded over a substantial portion of their length to assure flexibility of the connection. Tests have shown that the static strength of the connection is the same with or without end returns; therefore the use of returns is optional, but if used, their length must be restricted to not more than four times the weld size (Johnston and Green, 1940) (see Figure C-J2.8).

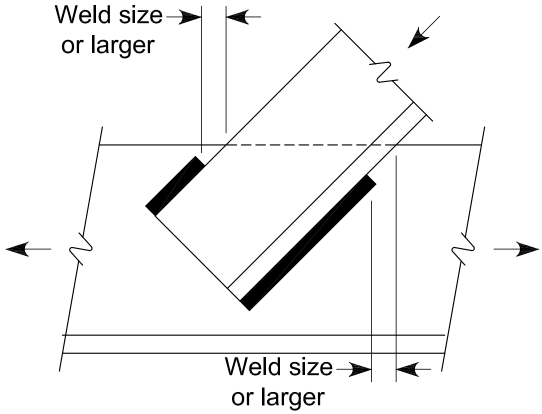


Fig. C-J2.5. Fillet welds near tension edges.

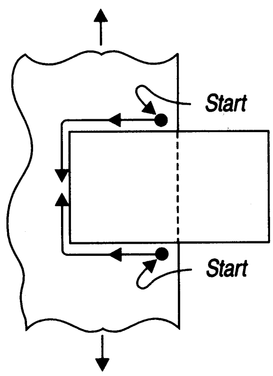


Fig. C-J2.6. Suggested direction of welding travel to avoid notches.

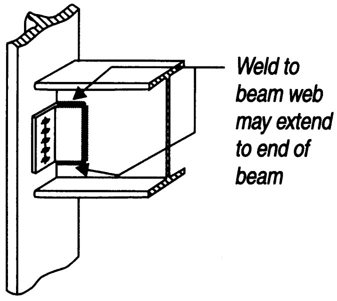


Fig. C-J2.7. Fillet weld details on framing angles.

(3) Experience has shown that when ends of intermediate transverse stiffeners on the webs of plate girders are not welded to the flanges (the usual practice), small torsional distortions of the flange occur near shipping bearing points in the normal course of shipping by rail or truck and may cause high out-of-plane bending stresses (up to the yield point) and fatigue cracking at the toe of the web-to-flange welds. This has been observed even with closely fitted stiffeners. The intensity of these out-of-plane stresses may be effectively limited and cracking prevented if "breathing room" is provided by terminating the stiffener weld away from the web-to-flange welds. The unwelded distance should not exceed six times the web thickness so that column buckling of the web within the unwelded length does not occur.

(4) For fillet welds that occur on opposite sides of a common plane, it is difficult to deposit a weld continuously around the corner from one side to the other without causing a gouge in the corner of the parts joined; therefore the welds must be interrupted at the corner (see Figure C-J2.9).

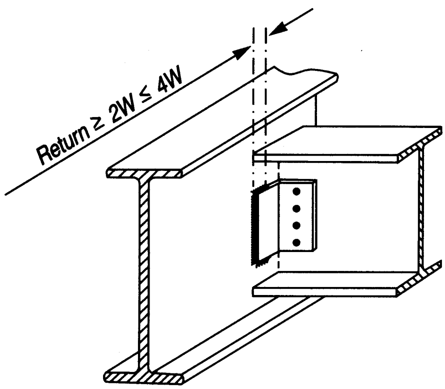


Fig. C-J2.8. Flexible connection returns optimal unless subject to fatigue.

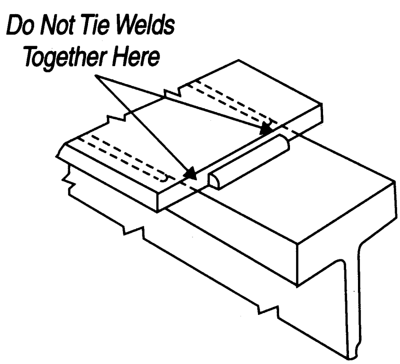


Fig. C-J2.9. Details for fillet welds that occur on opposite sides of a common plane.

3. Plug and Slot Welds

A plug weld is a weld made in a circular hole in one member of a joint fusing that member to another member. Both plug and slot welds are only applied to lap joints. Care should be taken when plug or slot welds are applied to structures subject to cyclic loading as the fatigue performance of these welds is limited. A slot weld is a weld made in an elongated hole in one member of a joint fusing that member to another member. A fillet weld inside a hole or slot is not a plug weld. A "puddle weld", typically used for joining decking to the supporting steel, is not the same as a plug weld.

3a. Effective Area

When plug and slot welds are detailed in accordance with Section J2.3b, the strength of the weld is controlled by the size of the fused area between the weld and the base metal. The total area of the hole or slot is used to determine the effective area.

3b. Limitations

Plug and slot welds are limited to situations where they are loaded in shear, or where they are used to prevent elements of a cross section from buckling, such as for web doubler plates on deeper rolled sections. Plug and slot welds are only allowed where the applied loads result in shear between the joined materials—they are not to be used to resist direct tensile loads.

The geometric limitations on hole and slot sizes are prescribed in order to provide a geometry that is conducive to good fusion. Deep, narrow slots and holes make it difficult for the welder to gain access and see the bottom of the cavity into which weld metal must be placed. Where access is difficult, fusion may be limited, and the strength of the connection reduced.

4. Strength

The strength of welds is governed by the strength of either the base material or the deposited weld metal. Table J2.5 presents the nominal weld strengths and the ϕ and Ω factors, as well as the limitations on filler metal strength levels.

The strength of a joint that contains a complete-joint-penetration (CJP) groove weld, whether loaded in tension or compression, is dependent upon the strength of the base metal, and no computation of the strength of the CJP groove weld is required. For tension applications, matching strength filler metal is required, as defined in AWS D1.1 Table 3.1. For compression applications, up to a 10 ksi (70 MPa) decrease in filler metal strength is permitted, which is equivalent to one strength level.

CJP groove welds loaded in tension or compression parallel to the weld axis, such as for the groove welded corners of box columns, do not transfer primary loads across the joint. In cases such as this, no computation of the strength of the CJP groove weld strength is required.

CJP groove welded tension joints are intended to provide strength equivalent to the base metal, therefore matching filler metal is required. CJP groove welds have been shown not to exhibit compression failure even when they are undermatched. The amount of undermatching before unacceptable deformation occurs has not been established, but one standard strength level is conservative and therefore permitted. Joints in which the weld strength is calculated based on filler metal classification strength can be designed using any filler metal strength equal to or less than matching. Filler metal selection is still subject to compliance with AWS D1.1.

The nominal strength of partial-joint-penetration (PJP) groove welded joints in compression is higher than for other joints because compression limit states are not observed on weld metal until significantly above the yield strength.

Connections that contain PJP groove welds designed to bear in accordance with Section J1.4(b), and where the connection is loaded in compression, are not limited in capacity by the weld since the surrounding base metal can transfer compression loads. When not designed in accordance with Section J1.4(b), an otherwise similar connection must be designed considering the possibility that either the weld or the base metal may be the critical component in the connection.

The factor of 0.6 on F_{EXX} for the tensile strength of PJP groove welds is an arbitrary reduction that has been in effect since the early 1960s to compensate for the notch effect of the unfused area of the joint, uncertain quality in the root of the weld due to the inability to perform nondestructive evaluation, and the lack of a specific notch-toughness requirement for filler metal. It does not imply that the tensile failure mode is by shear stress on the effective throat, as in fillet welds.

Column splices have historically been connected with relatively small PJP groove welds. Frequently, erection aids are available to resist construction loads. Columns are intended to be in contact bearing in splices and on base plates. Section M4.4 recognizes that, in the as-fitted product, the contact may not be consistent across the joint and therefore provides rules assuring some contact that limits the potential deformation of weld metal and the material surrounding it. These welds are intended to hold the columns in place, not to transfer the compressive loads. Additionally, the effects of very small deformation in column splices are accommodated by normal construction practices. Similarly, the requirements for base plates and normal construction practice assure some bearing at bases. Therefore the compressive stress in the weld metal does not need to be considered as the weld metal will deform and subsequently stop when the columns bear. Other PJP groove welded joints connect members that may be subject to unanticipated loads and may fit with a gap. Where these connections are finished to bear, fit-up may not be as good as that specified in Section M4.4 but some bearing is anticipated so the weld is to be designed to resist loads defined in Section J1.4(b) using the factors, strengths and effective areas in Table J2.5. Where the joints connect members that are not finished to bear, the welds are designed for the total required load using the available strengths, and areas in Table J2.5.

In Table J2.5 the nominal strength of fillet welds is determined from the effective throat area, whereas the strength of the connected parts is governed by their respective thicknesses. Figure C-J2.10 illustrates the shear planes for fillet welds and base material:

(1) Plane 1-1, in which the strength is governed by the shear strength of the material A.
(2) Plane 2-2, in which the strength is governed by the shear strength of the weld metal.
(3) Plane 3-3, in which the strength is governed by the shear strength of the material B.

The strength of the welded joint is the lowest of the strengths calculated in each plane of shear transfer. Note that planes 1-1 and 3-3 are positioned away from the fusion areas between the weld and the base material. Tests have demonstrated that the stress on this fusion area is not critical in determining the shear strength of fillet welds (Preece, 1968).

The shear planes for plug and PJP groove welds are shown in Figure C-J2.11 for the weld and base metal. Generally the base metal will govern the shear strength.

When weld groups are loaded in shear by an external load that does not act through the center of gravity of the group, the load is eccentric and will tend to cause a relative rotation and translation between the parts connected by the weld. The point about which rotation tends to take place is called the instantaneous center of rotation. Its location is dependent upon the load eccentricity, geometry of the weld group, and deformation of the weld at different angles of the resultant elemental force relative to the weld axis.

The individual strength of each unit weld element can be assumed to act on a line perpendicular to a ray passing through the instantaneous center and that element's location (see Figure C-J2.12).

The ultimate shear strength of weld groups can be obtained from the load deformation relationship of a single-unit weld element. This relationship was originally

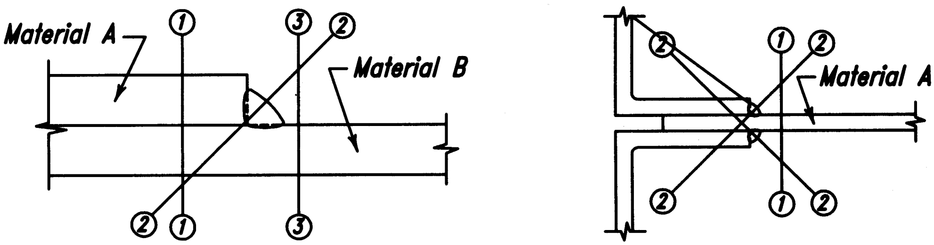


Fig. C-J2.10. Shear planes for fillet welds loaded in longitudinal shear.

given in Butler and others (1972) for E60 (E43) electrodes. Curves for E70 (E48) electrodes were reported in Lesik and Kennedy (1990).

Unlike the load-deformation relationship for bolts, strength and deformation performance in welds are dependent on the angle that the resultant elemental force makes with the axis of the weld element as shown in Figure C-J2.12. The actual load deformation relationship for welds is given in Figure C-J2.13, taken from Lesik and Kennedy (1990). Conversion of the SI equation to U.S. customary units results in the following weld strength equation for R_n:

$$R_n = 0.852(1.0 + 0.50 \sin^{1.5}\theta)F_{EXX}A_w \tag{C-J2-1}$$

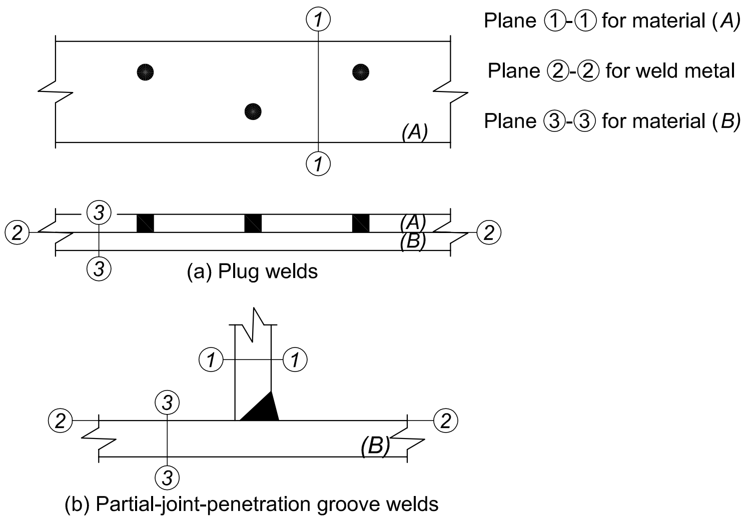


Fig. C-J2.11. Shear planes for plug and PJP groove welds.

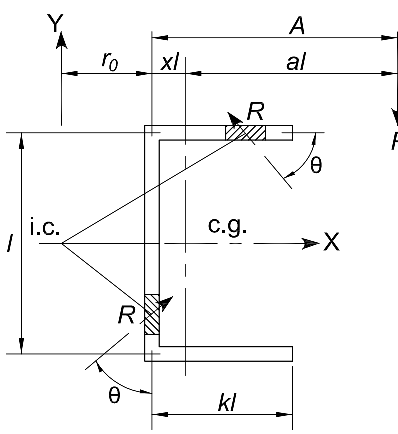


Fig. C-J2.12. Weld element nomenclature.

Because the maximum strength is limited to $0.60F_{EXX}$ for longitudinally loaded welds ($\theta = 0°$), the Specification provision provides, in the reduced equation coefficient, a reasonable margin for any variation in welding techniques and procedures. To eliminate possible computational difficulties, the maximum deformation in the weld elements is limited to $0.17w$. For design convenience, a simple elliptical formula is used for $f(p)$ to closely approximate the empirically derived polynomial in Lesik and Kennedy (1990).

The total strength of all the weld elements combine to resist the eccentric load and, when the correct location of the instantaneous center has been selected, the three in-plane equations of statics ($\Sigma F_x = 0$, $\Sigma F_y = 0$, $\Sigma M = 0$) will be satisfied. Numerical techniques, such as those given in Brandt (1982), have been developed to locate the instantaneous center of rotation subject to convergent tolerances.

5. Combination of Welds

When determining the capacity of a combination PJP groove weld and fillet weld contained within the same joint, the total throat dimension is not the simple addition of the fillet weld throat and the groove weld throat. In such cases, the resultant throat of the combined weld (dimension from root perpendicular to face of fillet weld) must be determined and the design based upon this dimension.

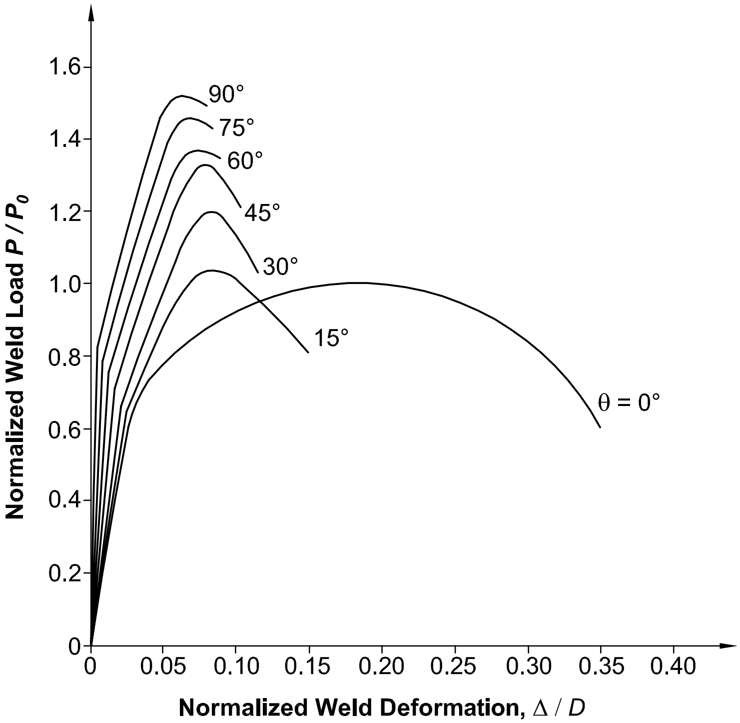


Fig. C-J2.13. Load deformation relationship.

6. Filler Metal Requirements

Applied and *residual stresses* and geometrical discontinuities from backup bars with associated notch effects contribute to sensitivity to fracture. Additionally, some weld metals in combination with certain procedures result in welds with low notch toughness. Accordingly, this Specification requires a minimum specified toughness for weld metals in those joints that are subject to more significant applied stresses and toughness demands. The level of toughness required is selected as one level more conservative than the base metal requirement for hot-rolled shapes with a flange thickness exceeding 2 in. (50 mm).

7. Mixed Weld Metal

Problems can occur when incompatible weld metals are used in combination and notch-tough composite weld metal is required. For instance, tack welds deposited using a self-shielded process with aluminum deoxidizers in the electrodes and subsequently covered by SAW weld passes can result in a composite weld metal with low notch-toughness, despite the fact that each process by itself could provide notch-tough weld metal.

Potential concern about intermixing weld metal types is limited to situations where one of the two weld metals is deposited by the self-shielded flux-cored arc welding (FCAW-s) process. Changes in tensile and elongation properties have been demonstrated to be of insignificant consequence. Notch toughness is the property that can be affected the most. Many compatible combinations of FCAW-s and other processes are commercially available.

J3. BOLTS AND THREADED PARTS

1. High-Strength Bolts

In general, the use of high-strength bolts is required to conform to the provisions of the *Specification for Structural Joints Using ASTM A325 or A490 Bolts* (RCSC, 2004) as approved by the Research Council on Structural Connections. Kulak (2002) provides an overview of the properties and use of high-strength bolts.

Occasionally the need arises for the use of high-strength bolts of diameters and lengths in excess of those available for ASTM A325 or A325M and ASTM A490 or A490M bolts. For joints requiring diameters in excess of $1^1/_2$ in. (38 mm) or lengths in excess of about 8 in. (200 mm), Section J3.1 permits the use of ASTM A449 bolts and ASTM A354 Grade BC and BD threaded rods. Note that anchor rods are more preferably specified as ASTM F1554 material.

Snug-tight installation is permitted for static applications involving only ASTM A325 or A325M bolts in tension or combined shear and tension. Two studies have been conducted to investigate possible reductions in strength because of varying levels of pretension in bolts within the same connection. The first investigation focused on nine, two-bolt tee stubs connected in a back-to-back configuration using $^3/_4$-in. diameter A325 bolts (Johnson, 1996). The bolt pretensions were varied from pretensioned to snug tight to finger tight. No significant loss of strength was noted as compared to the case with both fasteners pretensioned—even with one fastener pretensioned and the other finger tight. The second study tested 32 additional two-bolt tee stubs but considered both ASTM A325 and A490 fasteners and two, four-bolt tee stubs (Amrine and Swanson, 2004). The study found that no significant loss of strength resulted from having different pretensions in bolts within the same connection, even with ASTM A490 fasteners.

There are practical cases in the design of structures where slip of the connection is desirable to allow for expansion and contraction of a joint in a controlled manner. Regardless of whether force transfer is required in the direction normal to the slip direction, the nuts should be hand-tightened with a spud wrench and then backed off one-quarter turn. Furthermore, it is advisable to deform the bolt threads or use a locking nut or jamb nut to ensure that the nut does not back off further under service conditions. Thread deformation is commonly accomplished with a cold chisel and hammer applied at one location. Note that tack-welding of the nut to the bolt threads is not recommended.

2. Size and Use of Holes

To provide some latitude for adjustment in plumbing a frame during erection, three types of enlarged holes are permitted, subject to the approval of the designer. The nominal maximum sizes of these holes are given in Table J3.3 or J3.3M. The use of these enlarged holes is restricted to connections assembled with bolts and is subject to the provisions of Sections J3.3 and J3.4.

3. Minimum Spacing

The minimum spacing dimensions of $2^2/_3$ times and 3 times the nominal diameter are to facilitate construction and do not necessarily satisfy the bearing and tearout strength requirements in Section J3.10.

4. Minimum Edge Distance

The minimum edge distances given in Table J3.4 and Table J3.4M are to facilitate construction and do not necessarily satisfy the bearing and tearout strength requirements in Section J3.10. Lesser values are permitted if the requirements of Section J3.10 are satisfied.

5. Maximum Spacing and Edge Distance

Limiting the edge distance to not more than 12 times the thickness of an outside connected part, but not more than 6 in. (150 mm), is intended to provide for the exclusion of moisture in the event of paint failure, thus preventing corrosion between the parts that might accumulate and force these parts to separate. More restrictive limitations are required for connected parts of unpainted weathering steel exposed to atmospheric corrosion.

6. Tension and Shear Strength of Bolts and Threaded Parts

Tension loading of fasteners is usually accompanied by some bending due to the deformation of the connected parts. Hence, the resistance factor, ϕ, and the safety factor, Ω, are relatively conservative. The nominal tensile stress values in Table J3.2 were obtained from the equation

$$F_{nt} = 0.75F_u \tag{C-J3-2}$$

The factor of 0.75 included in this equation accounts for the approximate ratio of the effective area of the threaded portion of the bolt to the area of the shank of the bolt for common sizes. Thus A_b is defined as the area of the unthreaded body of the bolt and the value reported for F_{nt} in Table J3.2 is calculated as $0.75\ F_u$.

The tensile strength given by Equation C-J3-2 is independent of whether the bolt was initially installed pretensioned or snug-tightened. Recent tests confirm that the performance of ASTM A325 and A325M bolts in tension not subjected to fatigue are unaffected by the original installation condition (Amrine and Swanson, 2004; Johnson, 1996; Murray, Kline, and Rojani, 1992). While the equation was developed for bolted connections, it was also conservatively applied to threaded parts (Kulak and others, 1987).

For ASTM A325 or A325M bolts, no distinction is made between small and large diameters, even though the minimum tensile strength, F_u, is lower for bolts with diameters in excess of 1 in. (24 mm). It was felt that such a refinement was not justified, particularly in view of the conservative resistance factor, ϕ, and safety factor, Ω, the increasing ratio of tensile area to gross area, and other compensating factors.

The values of nominal shear stress in Table J3.2 were obtained from the following equations:

$$F_{nv} = 0.50F_u, \text{ when threads are excluded from the shear planes} \tag{C-J3-3}$$

$$F_{nv} = 0.40F_u, \text{ when threads are not excluded from the shear plane} \tag{C-J3-4}$$

The factors 0.50 and 0.40 account for the effect of shear and for the reduced area of the threaded portion of the fastener when the threads are not excluded from the shear plane. When determining the shear strength of a fastener, the area, A_b, is multiplied by the number of shear planes. While developed for bolted connections, the equations were also conservatively applied to threaded parts. The value given for ASTM A307 bolts was obtained from Equation C-J3-4 but is specified for all cases regardless of the position of threads.

In connections consisting of only a few fasteners, the effect of differential strain on the shear in bearing fasteners is negligible (Kulak and others, 1987; Fisher, Galambos, Kulak, and Ravindra, 1978). In longer joints, the differential strain produces an uneven distribution of load between fasteners, those near the end taking a disproportionate part of the total load, so that the maximum strength per fastener is reduced. This Specification does not limit the length but requires a 20 percent reduction in shear strength for connections longer than 50 in. (1.2 m). The resistance factor, ϕ, and the safety factor, Ω, for shear in bearing-type connections already accommodate the effects of differential strain in connections less than 50 in. (1.2 m) in length. The above discussion is primarily applicable to end-loaded connections, but is applied to all connections to maintain simplicity.

Additional information regarding the development of the provisions in this section can be found in the Commentary to the RCSC *Specification* (RCSC, 2004).

7. Combined Tension and Shear in Bearing-Type Connections

Tests have shown that the strength of bearing fasteners subject to combined shear and tension resulting from externally applied forces can be closely defined by an ellipse (Kulak and others, 1987). The relationship is expressed as

$$\left(\frac{f_t}{\phi F_{nt}}\right)^2 + \left(\frac{f_v}{\phi F_{nv}}\right)^2 = 1 \qquad \text{(LRFD)} \qquad \text{(C-J3-5a)}$$

$$\left(\frac{\Omega f_t}{F_{nt}}\right)^2 + \left(\frac{\Omega f_v}{F_{nv}}\right)^2 = 1 \qquad \text{(ASD)} \qquad \text{(C-J3-5b)}$$

In these equations, f_v and f_t are the required shear stress and tensile stress, respectively, and F_{nv} and F_{nt} are the nominal shear and tensile stresses, respectively. The elliptical relationship can be replaced, with only minor deviations, by three straight lines as shown in Figure C-J3.1. The sloped portion of the straight-line representation is

$$\left(\frac{f_t}{\phi F_{nt}}\right) + \left(\frac{f_v}{\phi F_{nv}}\right) = 1.3 \qquad \text{(LRFD)} \qquad \text{(C-J3-6a)}$$

$$\left(\frac{\Omega f_t}{F_{nt}}\right) + \left(\frac{\Omega f_v}{F_{nv}}\right) = 1.3 \qquad \text{(ASD)} \qquad \text{(C-J3-6b)}$$

which results in Equations J3-3a and J3-3b.

This latter representation offers the advantage that no modification of either type of stress is required in the presence of fairly large magnitudes of the other type. Note that Equations J3-3a and J3-3b can be rewritten so as to find the nominal shear strength per unit area, F'_{nv}, as a function of the required tensile stress, f_t. These formulations are

$$F'_{nt} = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v \le F_{nv} \qquad \text{(LRFD)} \qquad \text{(C-J3-7a)}$$

$$F'_{nt} = 1.3F_{nt} - \frac{\Omega F_{nt}}{F_{nv}} f_v \le F_{nt} \qquad \text{(ASD)} \qquad \text{(C-J3-7b)}$$

The linear relationship was adopted for use in Section J3.7; generally, use of the elliptical relationship is acceptable (see Figure C-J3.1). A similar formulation using the elliptical solution is

$$F'_{nt} = F_{nt}\sqrt{1-\left(\frac{f_v}{\phi F_{nv}}\right)^2} \le F_{nt} \qquad \text{(LRFD)} \qquad \text{(C-J3-8a)}$$

$$F'_{nt} = F_{nt}\sqrt{1-\left(\frac{\Omega f_{nv}}{F_{nv}}\right)^2} \le F_{nt} \qquad \text{(ASD)} \qquad \text{(C-J3-8b)}$$

8. High-Strength Bolts in Slip-Critical Connections

Connections should be classified as slip-critical only when the slip is deemed by the engineer of record to affect the serviceability of the structure by excessive distortion or cause a reduction in strength or stability even though the available strength of the connection is adequate. For example, connections subject to fatigue and connections with oversized holes or slots parallel to the direction of load should be designed as slip-critical. Most connections with standard holes can be designed as bearing-type connections without concern for serviceability. For connections with three or more bolts in standard holes or slots perpendicular to the direction of force, the freedom to slip generally does not exist because one or more of the bolts are in bearing before the load is applied.

Slip resistance in bolted connections has traditionally been viewed as a serviceability limit state and these connections have been designed to resist slip due to load effects from serviceability combinations and checked as bearing connections due to load effects from strength load combinations. There are conditions,

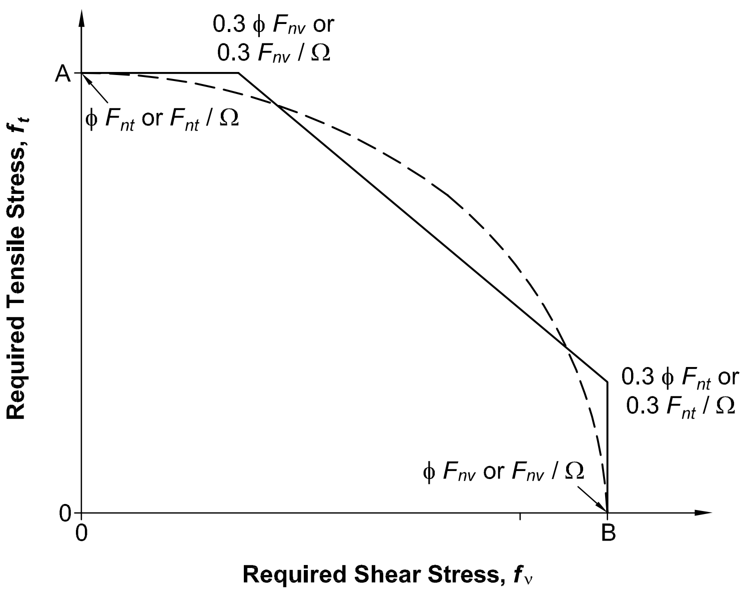


Fig. C-J3.1. Straight-line representation of elliptical solution.

however, where the deformations due to slip in connections with oversized holes and slotted holes parallel to the load could result in an increased load larger than the strength limit state. Examples where the usual assumption of serviceability-governed slip resistance may not apply are:

- High aspect ratio braced frames where the slip permitted by slots or oversized holes is relatively large and could potentially result in large P-Δ effects;
- Long-span, flat roof trusses with oversized holes, where slip could result in excessively large loads due to ponding;
- Built-up compression members where slip between the individual element ends could increase the member effective length and thus significantly reduce buckling strength;
- Any condition where the normal analysis assumption of an undeformed structure (small deflections) could be violated by connection slip resulting in increased load.

The Commentary to the 1999 *LRFD Specification* (AISC, 2000b) cautioned engineers about such conditions when utilizing long-slotted holes parallel to the direction of the load:

> If the connection is designed so that it will not slip under the effects of service loads, then the effect of the factored loads on the deformed structure (deformed by the maximum amount of slip in the long slots at all locations) must be included in the structural analysis. Alternatively, the connection can be designed so that it will not slip at loads up to the factored load level.

However, neither the 1999 *LRFD Specification* (AISC, 2000b) nor its Commentary provided the engineer with any guidance for designing to prevent slip at the factored load level. Since the AISC *LRFD Manual of Steel Construction*, 3rd edition (AISC, 2001) also provided two separate design aids, Tables 7-15 and 7-16, one that indicated the use of service load combinations and one that indicated the use of strength load combinations, it was sometimes believed that the use of Table 7-15 would guard against slip due to load effects from service load combinations and the use of Table 7-16 would guard against slip due to load effects from strength load combinations. These are incorrect interpretations as both tables lead to the same final result, that is, to prevent slip due to load effects from service load combinations.

The Commentary to the 1999 *LRFD Specification* (AISC, 2000b) states, "Slip of slip-critical connections is likely to occur at approximately 1.4 to 1.5 times the service load." This is based on the use of a resistance factor $\phi = 1.00$, standard holes, and calibrated wrench installation. The use of $\phi = 0.85$ for oversized and short-slotted holes and $\phi = 0.70$ for long-slotted holes perpendicular and $\phi = 0.60$ for long-slotted holes parallel to the load, increases this resistance to approximately 1.7 times the service load for oversized and short-slotted holes and even greater slip resistance for long-slotted holes. The use of the turn-of-the-nut installation method also increases slip resistance by approximately 10 to 15 percent. Hence connections with oversized and slotted holes, even when designed for the serviceability limit state provisions of the 1999 *LRFD Specification* (AISC, 2000b), will resist slip due to load effects from the strength load combinations.

Determining Required Resistance to Slip. This Specification permits all slip-critical connections with bolts in standard holes or in slotted holes perpendicular to the direction of the force to be designed as being governed by serviceability. The slight variations in geometry, which can occur due to connection slip when using this type of hole, will not change the normal analysis assumptions or result in an increase in load.

The fundamental design requirement for all connections with bolts in oversized holes and slotted holes parallel to the load is to prevent slip at the strength limit state, which conservatively assumes that the corresponding potential for change in geometry will not be negligible and that connection slip will result in significant load increase.

The engineer of record is permitted to make the determination that the effect of slip will not result in increased loads and, therefore, to design any slip-critical connection for the serviceability limit state. In either case, the design slip resistance is calculated using the load effects from either the LRFD load combinations or the ASD load combinations and the appropriate resistance factor, ϕ, or safety factor, Ω. All slip-critical connections, whether designed for the serviceability or strength limit state, must be checked for shear and bearing using the appropriate design loads.

The reliability required when designing to resist slip due to load effects from strength load combinations is subject to some interpretation. Traditionally, connection limit states require a β for bolts and fillet welds of 4.0. This is because many limit states associated with connection failure are associated with a sudden, nonductile joint separation. Since connection slip will not result in a sudden separation of the joint as long as the connection is checked as a bearing-type connection due to load effects from strength load combinations, knowing the exact level of reliability for slip resistance due to strength load combinations is not critical to connection performance. Resistance and safety factors along with the hole factors proposed for oversized holes and slotted holes approach those necessary to achieve a reliability index of 4.0. However, because of the complex factors involved in calculating the reliability of slip-critical connections and the lack of extensive statistical data on slip resistance of oversized and slotted holes, the checks for bearing and shear due to strength load combinations are required for both design methods.

Factors that Affect Slip Resistance of Joints. The following paragraphs outline the key factors affecting slip resistance in bolted steel connections:

Slip Coefficient of the Faying Surface. This Specification has combined the previous Class A and Class C surfaces into a single Class A surface category that includes unpainted clean mill scale surfaces or surfaces with Class A coatings on a blasted-cleaned surface, and hot-dip galvanized and roughened surfaces with a coefficient of friction $\mu = 0.35$. This is a slight increase in value from the previous Class A coefficient. Class B surfaces, unpainted blast-cleaned surfaces, or surfaces with Class B coatings on blast-cleaned steel remain the same at $\mu = 0.50$.

Pretensioning Method and D_u. Four bolt pretensioning methods are recognized by AISC: turn-of-the-nut, calibrated wrench, twist-off type tension-control bolt assemblies, and direct tension indicating assemblies. The mean pretension force in the bolts varies according to the method of installation. The lowest mean value is when the calibrated wrench method is used: 1.13 times the minimum specified. The turn-of-the-nut method results in a mean pretension of 1.22 to 1.35 times the minimum specified, depending on the amount the bolt is turned and the bolt grade. While the statistical information on the mean pretension level of bolts installed in the field using direct tension indicators and twist-off type tension-control bolt assemblies is limited, tests indicate they will fall somewhere between the calibrated wrench and the turn-of-the-nut method. Thus, this specification uses the minimum of these values, 1.13, for all methods of installation. This results in varying reliabilities for differing conditions. Regardless of the method used to pretension the bolts, it is important that the installation of slip-critical connections meet all of the requirements listed in the RCSC *Specification* (RCSC, 2004).

Hole Size. High-strength bolts properly installed in oversized and short-slotted holes using washers as specified in the RCSC *Specification* (RCSC, 2004) have the same resistance to slip as similar bolts in standard holes. The hole factor, $h_{sc} = 0.85$, is used to increase resistance to slip for this type of connection because of the possible consequences of increased movement with these connections. The hole factor for long-slotted holes, $h_{sc} = 0.70$, serves both to increase slip resistance for this type of connection similar to the oversized holes and to compensate for a slight loss in pretension and slip resistance due to the length of a long slot. Previous editions of the Specification had a further reduction in the hole factor, $h_{sc} = 0.60$, for slots parallel to the direction of the load. This was, in effect, a design for a strength limit state for this type of hole and the same result is achieved using the ϕ or Ω factors given in this Specification.

9. Combined Tension and Shear in Slip-Critical Connections

The slip resistance of a slip-critical connection is reduced if there is applied tension. The factor, k_s, is a multiplier that reduces the nominal slip resistance given by Equation J3-4 as a function of the applied tension load.

10. Bearing Strength at Bolt Holes

Provisions for bearing strength of pins differ from those for bearing strength of bolts; refer to Section J7.

Bearing strength values are provided as a measure of the strength of the material upon which a bolt bears, not as a protection to the fastener, which needs no such protection. Accordingly, the same bearing value applies to all joints assembled by bolts, regardless of fastener shear strength or the presence or absence of threads in the bearing area.

Material bearing strength may be limited either by bearing deformation of the hole or by tearout (a bolt-by-bolt block shear rupture) of the material upon which the bolt bears. Kim and Yura (1996) and Lewis and Zwerneman (1996) confirmed the bearing strength provisions for the bearing case wherein the nominal bearing strength R_n is equal to $CdtF_u$ and C is equal to 2.4, 3.0 or 2.0 depending upon hole type and/or acceptability of hole ovalization at ultimate load, as indicated in Section J3.10. However, this same research indicated the need for different bearing strength provisions when tearout failure would control. Appropriate equations for bearing strength as a function of clear distance L_c are therefore provided and this formulation is consistent with that in the RCSC *Specification* (RCSC, 2004).

Frank and Yura (1981) demonstrated that hole elongation greater than 1/4 in. (6 mm) will generally begin to develop as the bearing force is increased beyond $2.4dtF_u$, especially if it is combined with high tensile stress on the net section, even though rupture does not occur. For a long-slotted hole with the slot perpendicular to the direction of force, the same is true for a bearing force greater than $2.0dtF_u$. An upper bound of $3.0dtF_u$ anticipates hole ovalization [deformation greater than 1/4 in. (6 mm)] at maximum strength.

Additionally, to simplify and generalize such bearing strength calculations, the current provisions have been based upon a clear-distance formulation. Previous provisions utilized edge distances and bolt spacings measured to hole centerlines with adjustment factors to account for varying hole type and orientation, as well as minimum edge distance requirements.

11. Tension Fasteners

With any connection configuration where the fasteners transmit a tensile force to the HSS wall, a rational analysis must be used to determine the appropriate limit states. These may include a yield-line mechanism in the HSS wall and/or pull-out through the HSS wall, in addition to applicable limit states for the fasteners subject to tension.

J4. AFFECTED ELEMENTS OF MEMBERS AND CONNECTING ELEMENTS

Sections J4 and J5 of previous editions of the Specification have been combined in Section J4.

1. Strength of Elements in Tension

Tests have shown that yielding will occur on the gross section before the tensile capacity of the net section is reached if the ratio A_n/A_g is greater than or equal to

0.85 (Kulak and others, 1987). Since the length of connecting elements is small compared to the member length, inelastic deformation of the gross section is limited. Hence, the effective net area A_n of the connecting element is limited to $0.85A_g$ in recognition of the limited capacity for inelastic deformation, and to provide a reserve capacity.

2. Strength of Elements in Shear

In previous editions of the LRFD Specifications, the resistance factor for shear yielding had been 0.90, equivalent to a safety factor of 1.67. In ASD, the allowable shear yielding stress was $0.4F_y$, equivalent to a safety factor of 1.5. To make the LRFD approach in this Specification consistent with prior editions of the *ASD Specification*, the resistance and safety factors for shear yielding in this Specification are 1.0 and 1.5, respectively. The resulting increase in LRFD design strength of approximately 10 percent is justified by the long history of satisfactory performance of ASD use.

3. Block Shear Strength

Tests on coped beams indicated that a tearing failure mode (rupture) can occur along the perimeter of the bolt holes as shown in Figure C-J4.1 (Birkemoe and Gilmor, 1978). This block shear mode combines tensile failure on one plane and shear failure on a perpendicular plane. The failure path is defined by the centerlines of the bolt holes.

The block shear failure mode is not limited to coped ends of beams; other examples are shown in Figures C-J4.1 and C-J4.2. The block shear failure mode must also be checked around the periphery of welded connections.

This Specification has adopted a conservative model to predict block shear strength. The mode of failure in coped beam webs and angles is different than that of gusset plates because the shear resistance is present on only one plane, in which case there must be some rotation of the block of material that is providing the total resistance. Although tensile failure is observed through the net section

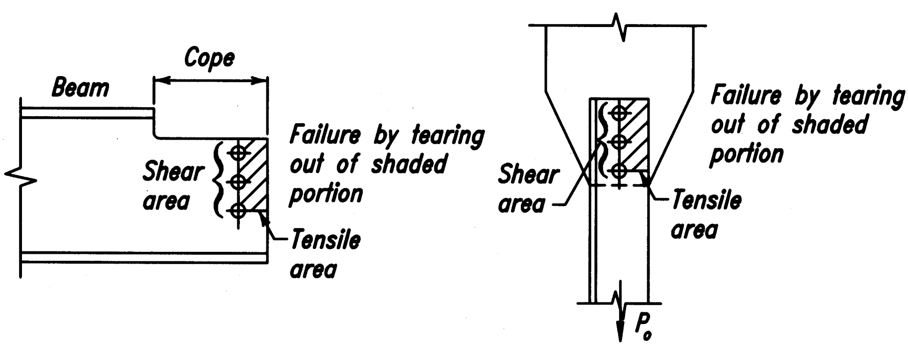


Fig. C-J4.1. Failure surface for block shear rupture limit state.

on the end plane, the distribution of tensile stresses is not always uniform (Ricles and Yura, 1983; Kulak and Grondin, 2001a). A reduction factor, U_{bs}, has been included in Equation J4-5 to approximate the non-uniform stress distribution on the tensile plane. The tensile stress distribution is non-uniform in the two row connection in Figure C-J4.2(b) because the rows of bolts nearest the beam end pick up most of the shear load.

Block shear is a rupture or tearing phenomenon, not a yielding limit state. However, gross yielding on the shear plane can occur when tearing on the tensile plane commences if $0.6F_uA_{nv}$ exceeds $0.6F_yA_{gv}$. Hence, Equation J4-5 limits the term $0.6F_yA_{gv}$ to not greater than $0.6F_uA_{nv}$. Equation J4-5 is consistent with the philosophy in Chapter D for tension members where the gross area is used for the limit state of yielding and the net area is used for the limit state of rupture.

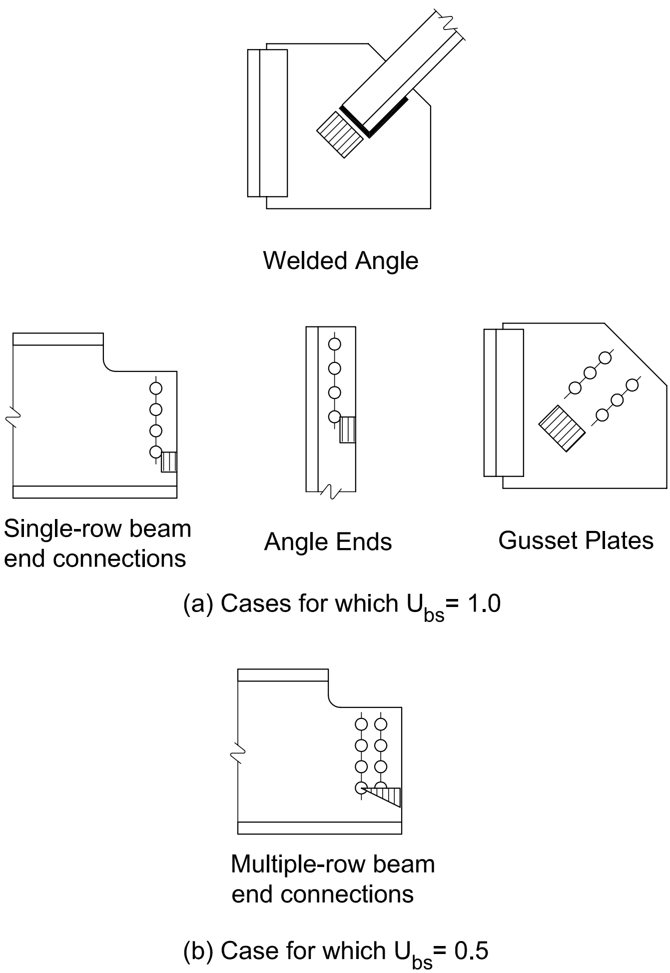


Fig. C-J4.2. Block shear tensile stress distributions.

4. Strength of Elements in Compression

To simplify connection calculations, the nominal strength of elements in compression when the element slenderness ratio is not greater than 25 is $F_y A_g$, which is a very slight increase over that obtained if the provisions of Chapter F are used. For more slender elements, the provisions of Chapter F apply.

J5. FILLERS

The practice of securing fillers by means of additional fasteners, so that they are, in effect, an integral part of a shear-connected component, is not required where a connection is designed for slip at member required strength levels. In such connections, the resistance to slip between the filler and either connected part is comparable to that which would exist between the connected parts if no filler were present.

Filler plates may be used in lap joints of welded connections that splice parts of different thickness, or where there may be an offset in the joint.

J6. SPLICES

The nominal strength of the smaller plate must be developed when groove-welded splices are used in plate girders and beams. For other connections it is sufficient to provide a connection to resist the required force at the joint.

J7. BEARING STRENGTH

In general, the bearing strength design of milled surfaces is governed by the limit state of bearing (local compressive yielding) at nominal loads, resulting in a stress of $0.9F_y$. Adequate safety is provided by post-yield strength as deformation increases. Tests on pin connections (Johnston, 1939) and on rockers (Wilson, 1934) have confirmed this behavior.

As used throughout the Specification, the terms "milled surface," "milled" and "milling" are intended to include surfaces that have been accurately sawed or finished to a true plane by any suitable means.

J8. COLUMN BASES AND BEARING ON CONCRETE

The provisions of this section are identical to equivalent provisions in ACI 318 (ACI, 2002).

J9. ANCHOR RODS AND EMBEDMENTS

The term "anchor rod" is used for threaded rods embedded in concrete to anchor structural steel. The term "rod" is intended to clearly indicate that these are threaded rods, not structural bolts, and should be designed as threaded parts per Table J3.2 using the material specified in Section A3.4.

Generally, the largest tensile force for which anchor rods need to be designed is that produced by bending moment at the column base and augmented by any uplift caused by the overturning tendency of a building under lateral load.

Shear at the base of a column is seldom resisted by bearing of the column base plate against the anchor rods. Even considering the lowest conceivable slip coefficient, the friction due to the vertical load on a column is generally more than sufficient to result in the transfer by frictional resistance of any likely amount of shear from the column base to the foundation. The possible exception is at the base of braced frames and moment frames where larger shear forces may require that shear transfer be accomplished by embedding the column base or providing a shear key at the top of the foundation.

The anchor rod hole sizes listed in Tables C-J9.1 and C-J9.1M are recommended to accommodate the tolerance required for setting anchor rods cast in concrete. These larger hole sizes are not detrimental to the integrity of the supported structure when used with proper washers. The slightly conical hole that results from punching operations or thermal cutting is acceptable.

If plate washers are utilized to resolve horizontal shear, bending in the anchor rod must be considered in the design and the layout of anchor rods must accommodate plate washer clearances. In this case special attention must be given to weld clearances, accessibility, edge distances on plate washers, and the effect of the tolerances between the anchor rod and the edge of the hole.

It is important that the placement of anchor rods be coordinated with the placement and design of reinforcing steel in the foundations as well as the design and overall size of base plates. It is recommended that the anchorage device at the anchor rod bottom be as small as possible to avoid interference with the reinforcing steel in the foundation. A heavy-hex nut or forged head is adequate to develop the concrete shear cone. See DeWolf and Ricker (1990) for design of base plates and anchor rods along with ACI 318 (ACI, 2002) and ACI 349 (ACI, 2001) for embedment design. Also see OSHA *Safety and Health Regulations for Construction*, Standards—29 CFR 1926 Subpart R—Steel Erection (OSHA, 2001) for anchor rod design and construction requirements for erection safety.

J10. FLANGES AND WEBS WITH CONCENTRATED FORCES

This Specification separates flange and web strength requirements into distinct categories representing different limit states, namely, flange local bending (Section J10.1), web local yielding (Section J10.2), web crippling (Section J10.3), web *sidesway buckling* (Section J10.4), web compression buckling (Section J10.5), and web panel-zone shear (Section J10.6).

These limit state provisions are applied to two distinct types of concentrated forces normal to member flanges:

TABLE C-J9.1
Anchor Rod Hole Diameters, in.

Anchor Rod Diameter	Anchor Rod Hole Diameter
$^1/_2$	$1^1/_{16}$
$^5/_8$	$1^3/_{16}$
$^3/_4$	$1^5/_{16}$
$^7/_8$	$1^9/_{16}$
1	$1^{13}/_{16}$
$1^1/_4$	$2^1/_{16}$
$1^1/_2$	$2^5/_{16}$
$1^3/_4$	$2^3/_4$
≥ 2	$d_b + 1^1/_4$

TABLE C-J9.1M
Anchor Rod Hole Diameters, mm.

Anchor Rod Diameter	Anchor Rod Hole Diameter
18	32
22	36
24	42
27	48
30	51
33	54
36	60
39	63
42	74

Single concentrated forces may be tensile (such as those delivered by tension hangers) or compressive (such as those delivered by bearing plates at beam interior positions, reactions at beam ends, and other bearing connections). Flange local bending applies only for tensile forces, web local yielding applies to both tensile and compressive forces, and the remainder of these limit states apply only to compressive forces. Double concentrated forces, one tensile and one compressive, form a couple on the same side of the loaded member, such as that delivered to column flanges through welded and bolted moment connections.

Transverse stiffeners, also called continuity plates, and web doubler plates are only required when the demand (the transverse concentrated force) exceeds the available strength. It is often more economical to choose a heavier member than to provide such reinforcement (Carter, 1999; Troup, 1999). The demand may be determined as the largest flange force from the various load cases, although the demand may also be taken as the gross area of the attachment delivering the force multiplied by the specified minimum yield strength, F_y. Stiffeners and/or doublers and their attaching welds are sized for the difference between the demand and the applicable limit state strength. Requirements for stiffeners are provided in Sections J10.7 and J10.8 and requirements for doublers are provided in Section J10.9.

1. Flange Local Bending

Where a tensile force is applied through a plate welded across a flange, that flange must be sufficiently rigid to prevent deformation of the flange and the corresponding high stress concentration in the weld in line with the web.

The effective column flange length for local flange bending is $12t_f$ (Graham, Sherbourne, Khabbaz, and Jensen, 1960). Thus, it is assumed that yield lines form in the flange at $6t_f$ in each direction from the point of the applied concentrated force. To develop the fixed edge consistent with the assumptions of this model, an additional $4t_f$, and therefore a total of $10t_f$, is required for the full flange-bending strength given by Equation J10-1. In the absence of applicable research, a 50 percent reduction has been introduced for cases wherein the applied concentrated force is less than $10t_f$ from the member end.

The strength given by Equation J10-1 was originally developed for moment connections but also applies to single concentrated forces such as tension hangers consisting of a plate welded to the bottom flange of a beam and transverse to the beam web. In the original tests, the strength given by Equation J10-1 was intended to provide a lower bound to the force required for weld fracture, which was aggravated by the uneven stress and strain demand on the weld caused by the flange deformation (Graham, Sherbourne, and Khabbaz, 1959).

Recent tests on welds with minimum Charpy V-Notch (CVN) toughness requirements show that weld fracture is no longer the failure mode when the strength given by Equation J10-1 is exceeded. Rather, it was found that the strength given by Equation J10-1 is consistently less than the force required to separate the flanges in typical column sections by $^1/_4$ in. (6 mm) (Hajjar, Dexter, Ojard, Ye, and Cotton, 2003; Prochnow, Ye, Dexter, Hajjar, and Cotton, 2000). This amount of flange deformation is on the order of the tolerances in ASTM A6, and it is believed that if the flange deformation exceeded this level it could be detrimental to other aspects of the performance of the member, such as flange local buckling. Although this deformation could also occur under compressive normal forces, it is customary that flange local bending is checked only for tensile forces (because the original concern was weld fracture). Therefore it is not required to check flange local bending for compressive forces.

The provision in Section J10.1 is not applicable to moment end-plate and tee-stub type connections. For these connections, see Carter (1999) or the AISC *Manual of Steel Construction* (AISC, 2005a).

2. Web Local Yielding

The web local yielding provisions (Equations J10-2 and J10-3) apply to both compressive and tensile forces of bearing and moment connections. These provisions are intended to limit the extent of yielding in the web of a member into which a force is being transmitted. The provisions are based on tests on two-sided directly welded girder-to-column connections (cruciform tests) (Sherbourne and Jensen, 1957) and were derived by considering a stress zone that spreads out with a slope of 2:1. Graham and others (1960) report pull-plate tests and suggest that a 2.5:1 stress gradient would be more appropriate.

Recent tests confirm that the provisions given by Equations J10-2 and J10-3 are slightly conservative and that the yielding is confined to a length consistent with the slope of 2.5:1 (Hajjar and others, 2003; Prochnow and others, 2000).

3. Web Crippling

The web crippling provisions (Equations J10-4 and J10-5) apply only to compressive forces. Originally, the term “web crippling” was used to characterize phenomena now called local web yielding, which was then thought to also adequately predict web crippling. The first edition of the AISC *LRFD Specification* (AISC, 1986) was the first AISC Specification to distinguish between local web yielding and local web crippling. Web crippling was defined as crumpling of the web into buckled waves directly beneath the load, occurring in more slender webs, whereas web local yielding is yielding of that same area, occurring in stockier webs.

Equations J10-4 and J10-5 are based on research reported in Roberts (1981). The increase in Equation J10-5b for $N/d > 0.2$ was developed after additional testing to better represent the effect of longer bearing lengths at ends of members (Elgaaly and Salkar, 1991). All tests were conducted on bare steel beams without the expected beneficial contributions of any connection or floor attachments. Thus, the resulting provisions are considered conservative for such applications. Kaczinski, Schneider, Dexter, and Lu (1994) reported tests on cellular box beams with slender webs and confirmed that these provisions are appropriate in this type of structure as well.

The equations were developed for bearing connections but are also generally applicable to moment connections.

The web crippling phenomenon has been observed to occur in the web adjacent to the loaded flange. For this reason, a half-depth stiffener (or stiffeners) or a half-depth doubler plate is needed to eliminate this limit state.

4. Web Sidesway Buckling

The web *sidesway buckling* provisions (Equations J10-6 and J10-7) apply only to compressive forces in bearing connections and do not apply to moment connections. The web *sidesway buckling* provisions were developed after observing several unexpected failures in tested beams (Summers and Yura, 1982; Elgaaly, 1983). In those tests the compression flanges were braced at the concentrated load, the web was subjected to compression from a concentrated load applied to the flange and the tension flange buckled (see Figure C-J10.1).

Web *sidesway buckling* will not occur in the following cases:

(a) For flanges restrained against rotation (such as when connected to a slab), when

$$\frac{h/t_w}{l/b_f} > 2.3 \tag{C-J10-1}$$

(b) For flanges *not* restrained against rotation, when

$$\frac{h/t_w}{l/b_f} > 1.7 \tag{C-J10-2}$$

where l is as shown in Figure C-J10.2.

Web *sidesway buckling* can be prevented by the proper design of lateral bracing or stiffeners at the load point. It is suggested that local bracing at both flanges be designed for 1 percent of the concentrated force applied at that point. If stiffeners are used, they must extend from the load point through at least one-half the beam or girder depth. In addition, the pair of stiffeners must be designed to carry the full load. If flange rotation is permitted at the loaded flange, neither stiffeners nor doubler plates are effective.

5. Web Compression Buckling

The web compression buckling provision (Equation J10-8) applies only when there are compressive forces on both flanges of a member at the same cross section, such as might occur at the bottom flange of two back-to-back moment connections under gravity loads. Under these conditions, the member web must have its slenderness ratio limited to avoid the possibility of buckling. Equation J10-8 is applicable to a pair of moment connections, and to other pairs of compressive forces applied at both flanges of a member, for which N/d is approximately less than 1. When N/d is not small, the member web should be designed as a compression member in accordance with Chapter E.

Equation J10-8 is predicated on an interior member loading condition. In the absence of applicable research, a 50 percent reduction has been introduced for cases wherein the compressive forces are close to the member end.

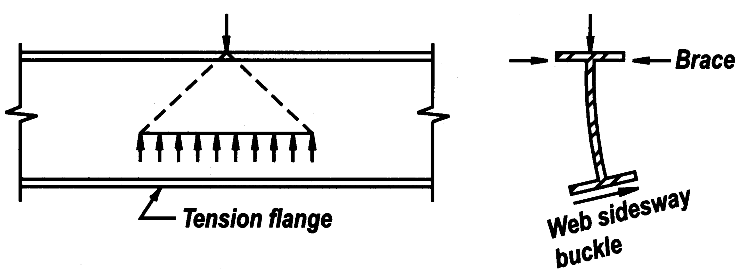


Fig. C-J10.1. Web sidesway buckling.

6. Web Panel-Zone Shear

Column web shear stresses may be significant within the boundaries of the rigid connection of two or more members with their webs in a common plane. Such webs must be reinforced when the required force ΣF_u for LRFD or ΣF for ASD along plane A-A in Figure C-J10.3 exceeds the column web available strength ϕR_v or R_v / Ω, respectively, where

for LRFD

$$\Sigma F_u = \frac{M_{u1}}{d_{m1}} + \frac{M_{u2}}{d_{m2}} - V_u \qquad \text{(C-J10-3a)}$$

and

$M_{u1} = M_{u1L} + M_{u1G}$ = sum of the moments due to the factored lateral loads, M_{u1L}, and the moments due to factored gravity loads, M_{u1G}, on the windward side of the connection, kip-in. (N-mm)

$M_{u2} = M_{u2L} - M_{u2G}$ = difference between the moments due to the factored lateral loads M_{u2L} and the moments due to factored gravity loads, M_{u2G}, on the windward side of the connection, kip-in. (N-mm)

for ASD

$$\Sigma F = \frac{M_{a1}}{d_{m1}} + \frac{M_{a2}}{d_{m2}} - V \qquad \text{(C-J10-3b)}$$

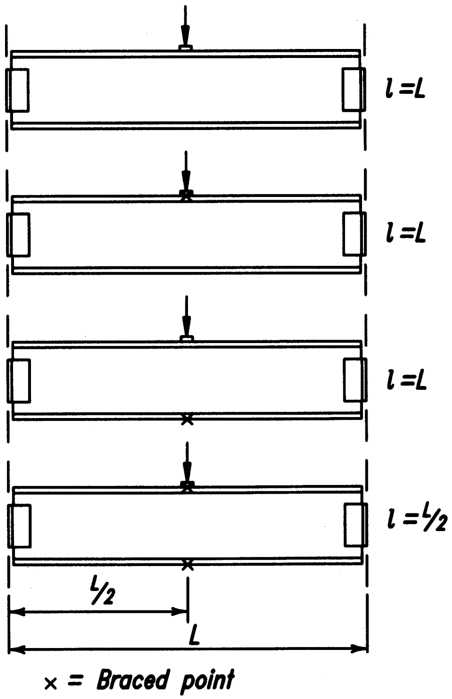


Fig. C-J10.2. Unbraced flange length for web sidesway buckling.

and

$M_{a1} = M_{a1L} + M_{a1G}$ = sum of the moments due to the nominal lateral loads, M_{a1L}, and the moments due to nominal gravity loads, M_{aG}, on the windward side of the connection, kip-in. (N-mm)

$M_{a2} = M_{a2L} + M_{a2G}$ = difference between the moments due to the nominal lateral loads, M_{a2L}, and the moments due to nominal gravity loads, M_{a2G}, on the windward side of the connection, kip-in. (N-mm)

d_{m1}, d_{m2} = distance between flange forces in the moment connection, in. (mm)

Historically (and conservatively), 0.95 times the beam depth has been used for d_m.

If, for LRFD $\Sigma F_u \leq \phi R_v$, or for ASD $\Sigma F \leq R_v / \Omega$, no reinforcement is necessary, in other words, $t_{req} \leq t_w$, where t_w is the column web thickness.

Equations J10-9 and J10-10 limit panel-zone behavior to the elastic range. While such connection panels possess large reserve capacity beyond initial general shear yielding, the corresponding inelastic joint deformations may adversely affect the strength and stability of the frame or story (Fielding and Huang, 1971; Fielding and Chen, 1973). Panel-zone shear yielding affects the overall frame stiffness and, therefore, the resulting second-order effects may be significant. The shear/axial interaction expression of Equation J10-10, as shown in Figure C-J10.4, provides elastic panel behavior.

If adequate connection ductility is provided and the frame analysis considers the inelastic panel-zone deformations, then the additional inelastic shear strength is recognized in Equations J10-11 and J10-12 by the factor

$$\left(1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_w}\right)$$

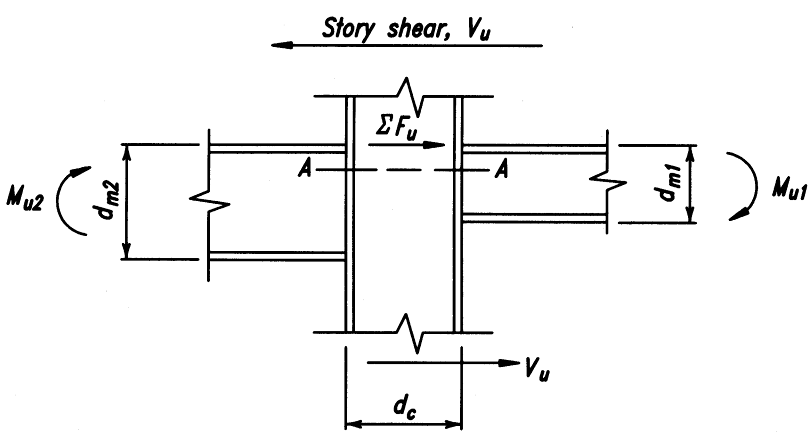


Fig. C-J10.3. LRFD forces in panel zone (ASD forces are similar).

This inelastic shear strength has been most often utilized for the design of frames in high seismic design and should be used when the panel zone is designed to develop the strength of the members from which it is formed.

The shear/axial interaction expression incorporated in Equation J10-12 (see Figure C-J10.5) recognizes that when the panel-zone web has completely yielded in shear, the axial column load is carried in the flanges.

7. Unframed Ends of Beams and Girders

Full-depth stiffeners are required at unframed ends of beams and girders not otherwise restrained to avoid twisting about their longitudinal axes.

8. Additional Stiffener Requirements for Concentrated Forces

See Carter (1999), Troup (1999), and Murray and Sumner (2004) for guidelines on column stiffener design.

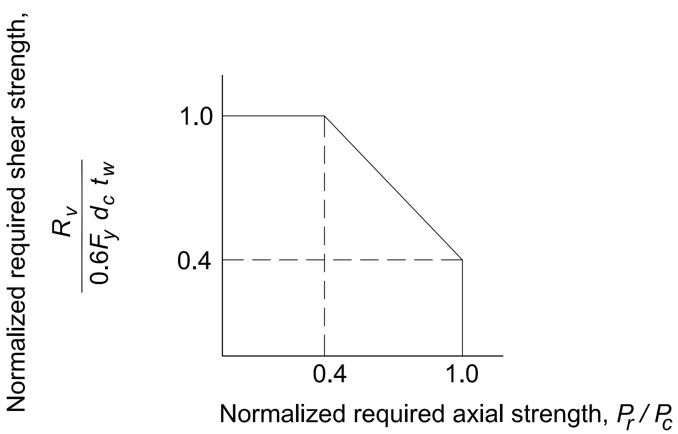


Fig. C-J10.4. Interaction of shear and axial force—elastic.

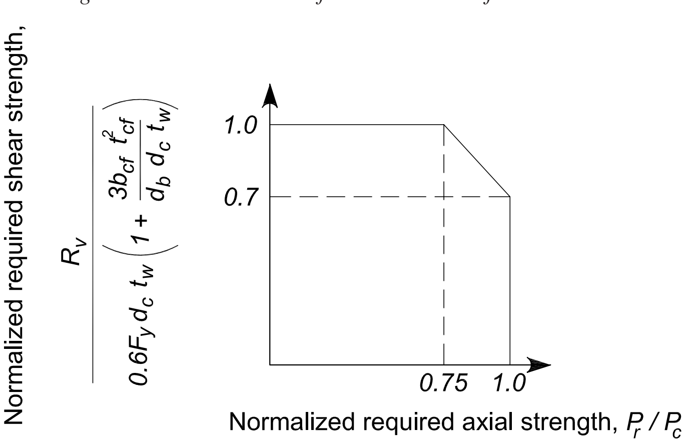


Fig. C-J10.5. Interaction of shear and axial force—inelastic.

For rotary straightened W-shapes, an area of reduced notch toughness is sometimes found in a limited region of the web immediately adjacent to the flange, referred to as the "k-area," as illustrated in Figure C-J10.6 (Kaufmann, Metrovich, Pense, and Fisher, 2001). Following the 1994 Northridge Earthquake, there was a tendency to specify thicker continuity plates that were groove welded to the web and flange and thicker doubler plates that were often groove welded in the gap between the doubler plate and the flanges. These welds were highly restrained and may have caused cracking during fabrication in some cases (Tide, 1999).

AISC (1997a) recommended that the welds for continuity plates should terminate away from the k-area, which is defined as the "region extending from approximately the midpoint of the radius of the fillet into the web approximately 1 to 1½ in. (25 to 38 mm) beyond the point of tangency between the fillet and web."

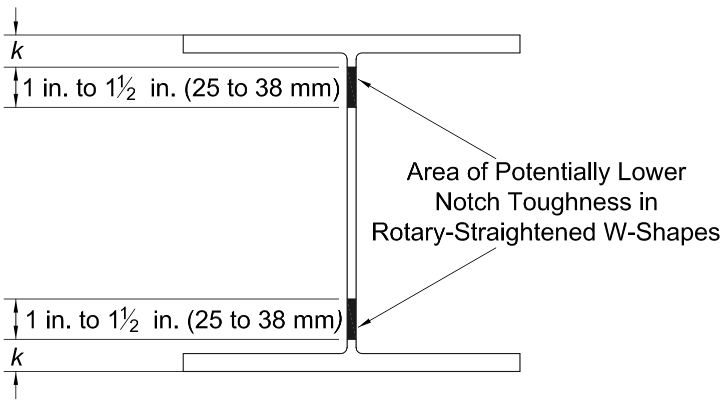


Fig. C-J10.6. Representative "k-area" of a wide-flange shape.

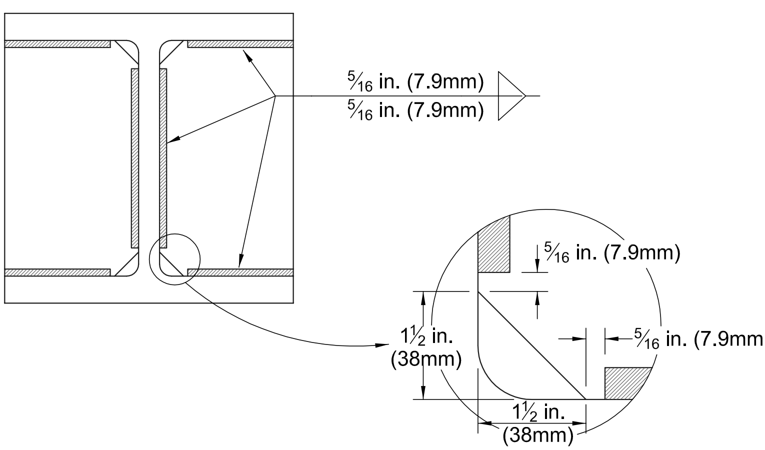


Fig. C-J10.7. Recommended placement of stiffener fillet welds to avoid contact with "k-area."

Recent pull-plate testing (Dexter and Melendrez, 2000; Prochnow and others, 2000; Hajjar and others, 2003) and full-scale beam-column joint testing (Bjorhovde, Goland, and Benac, 1999; Dexter, Hajjar, Prochnow, Graeser, Galambos, and Cotton, 2001; Lee, Cotton, Dexter, Hajjar, Ye, and Ojard, 2002) has shown that this problem can be avoided if the column stiffeners are fillet welded to both the web and the flange, the corner is clipped at least 1½ in. (38 mm), and the fillet welds are stopped short by a weld leg length from the edges of the cutout, as shown in Figure C-J10.7. These tests also show that groove welding the stiffeners to the flanges or the web is unnecessary, and that the fillet welds performed well with no problems. If there is concern regarding the development of the stiffeners using fillet welds, the corner clip can be made so that the dimension along the flange is ¾ in. (20 mm) and the dimension along the web is 1½ in. (38 mm).

Recent tests have also shown the viability of fillet welding doubler plates to the flanges, as shown in Figure C-J10.8 (Prochnow and others, 2000; Dexter and others, 2001; Lee and others, 2002; Hajjar and others, 2003). It was found that it

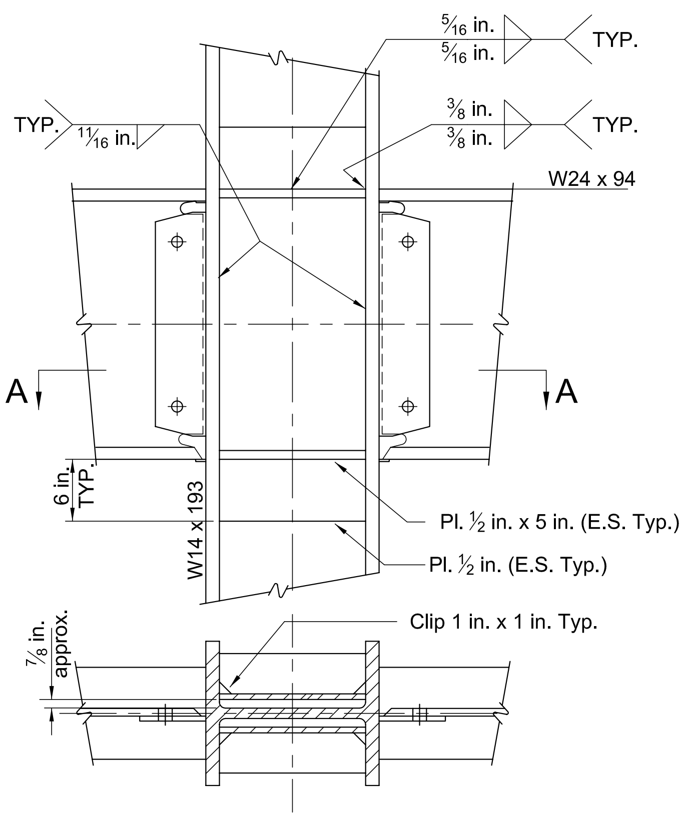


Fig. C-J10.8. Example of fillet welded doubler plate and stiffener details.

is not necessary to groove weld the doubler plates and that they do not need to be in contact with the column web to be fully effective.

9. Additional Doubler Plate Requirements for Concentrated Forces

When required, doubler plates are to be designed using the appropriate limit state requirements for the type of loading. The sum of the strengths of the member element and the double plate(s) must exceed the required strength and the doubler plate must be welded to the member element.

CHAPTER K

DESIGN OF HSS AND BOX MEMBER CONNECTIONS

Chapter K addresses the strength of HSS and box member welded connections. The provisions are based upon failure modes that have been reported in international research on HSS, much of which has been sponsored and synthesized by CIDECT (International Committee for the Development and Study of Tubular Construction) since the 1960s. This work has also received critical review by the International Institute of Welding (IIW) Subcommission XV-E on "Welded Joints in Tubular Structures." The HSS connection design recommendations are generally in accord with the last edition of the design recommendations by this Subcommission (IIW, 1989). Some minor modifications to the IIW recommended provisions for some limit states have been made by the adoption of the formulations for the same limit states elsewhere in this Specification. The IIW connection design recommendations referred to above have also been implemented and supplemented in later design guides by CIDECT (Wardenier, Kurobane, Packer, Dutta, and Yeomans, 1991; Packer, Wardenier, Kurobane, Dutta, and Yeomans, 1992), in the design guide by the Canadian Institute of Steel Construction (Packer and Henderson, 1997) and in Eurocode 3 (2002). Parts of these IIW design recommendations are also incorporated in AWS (2004). A large amount of research data generated by CIDECT research programs up to the mid-1980s is summarized in CIDECT Monograph No. 6 (Giddings and Wardenier, 1986). Further information on CIDECT publications and reports can be obtained from their website: *www.cidect.com.*

The scopes of Sections K2 and K3 note that the centerlines of the branch member(s) and the chord members must lie in a single plane. For other configurations, such as multiplanar connections, connections with partially or fully flattened branch member ends, double chord connections, connections with a branch member that is offset so that its centerline does not intersect with the centerline of the chord or connections with round branch members joined to a square or rectangular chord member, the provisions of IIW (1989), CIDECT, Wardenier and others (1991), Packer and others (1992), CISC, Packer and Henderson (1997), Marshall (1992), AWS (2004), or other verified design guidance or tests can be used.

K1. CONCENTRATED FORCES ON HSS

1. Definitions of Parameters

Some of the notation used in Chapter K is illustrated in Figure C-K1.1.

2. Limits of Applicability

The limits of applicability in Section K1.2 stem primarily from limitations on tests conducted to date.

3. Concentrated Force Distributed Transversely

Sections K1.3 and K1.4, although pertaining to all concentrated forces on HSS, are particularly oriented towards plate-to-HSS welded connections and this application is displayed in tabular form in Table C-K1.1 (a) and (b). In addition to the design provisions in the Specification, Table C-K1.1(b) also gives flexural strengths for some plate-to-round HSS connections. Most of the equations (after application of appropriate resistance factors for LRFD) conform to CIDECT Design Guides 1 and 3 (Wardenier and others, 1991; Packer and others, 1992) with updates in accordance with CIDECT Design Guide 9 (Kurobane, Packer, Wardenier, and Yeomans, 2004). The latter includes revisions for longitudinal plate-to-rectangular HSS connections (Equation K1-9) based on extensive experimental and numerical studies reported in Kosteski and Packer (2003). The provisions for the limit state of sidewall crippling of rectangular HSS, Equations K1-5 and K1-6, conform to web crippling expressions elsewhere in this Specification, and not to CIDECT or IIW recommendations. If a longitudinal plate-to-rectangular HSS connection is made by passing the plate through a slot in the HSS and then welding the plate to both the front and back HSS faces to form a "through-plate connection," the nominal strength can be taken as twice that given by Equation K1-9 (Kosteski and Packer, 2003).

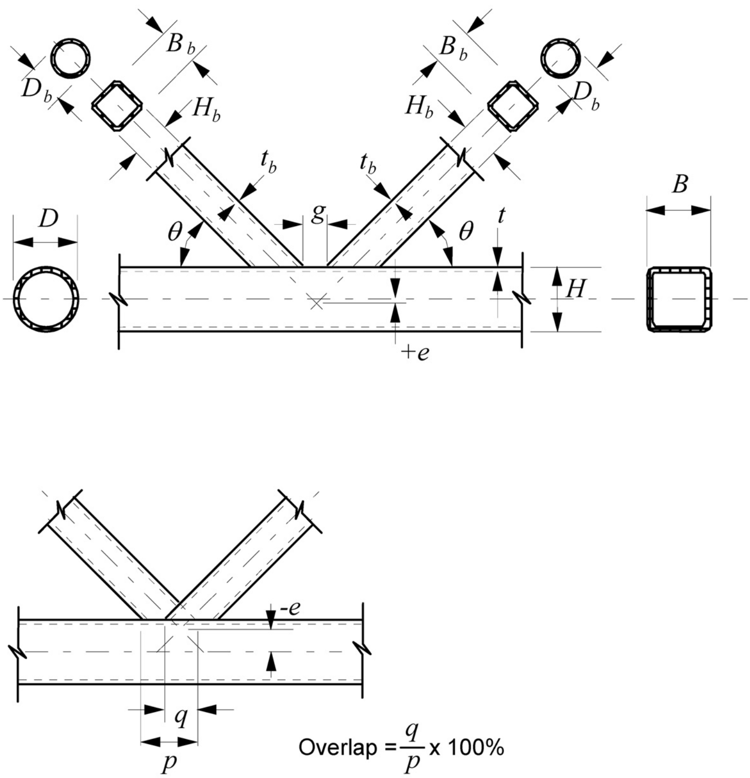


Fig. C-K1.1. Common notation for HSS connections.

The equations given for transverse plate-to-HSS connections can also be adapted for wide-flange beam-to-HSS PR moment connections, by treating the beam flanges as a pair of transverse plates and ignoring the beam web. For such wide-flange beam connections, the beam moment is thus produced by a force couple in the beam flanges. The connection flexural strength is then given by the plate-to-HSS connection strength multiplied by the distance between the beam flange centers. In Table C-K1.1(a) there is no check for the limit state of chord wall plastification for transverse plate-to-rectangular HSS connections, because this will not govern the design in practical cases. However, if there is a major compression load in the HSS, such as when it is used as a column, one should be aware that this compression load in the main member has a negative influence on the yield line plastification failure mode of the connecting chord wall (via a Q_f factor). In such a case, the designer can utilize guidance in CIDECT Design Guide No. 9 (Kurobane and others, 2004).

4. Concentrated Force Distributed Longitudinally at the Center of the HSS Diameter or Width, and Acting Perpendicular to the HSS Axis

See commentary for Section K1.3.

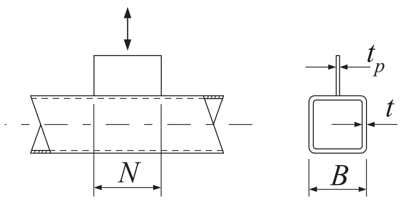
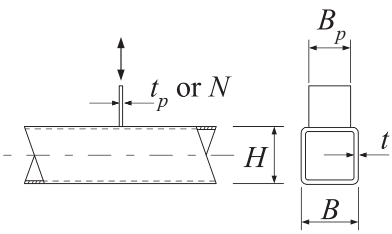
5. Concentrated Force Distributed Longitudinally at the Center of the HSS Width, and Acting Parallel to the HSS Axis

Section K1.5 applies to longitudinal plate connections loaded in shear. These recommendations are based on Sherman and Ales (1991), Sherman (1995a) and Sherman (1996) that investigated a large number of simple framing connections between wide-flange beams and rectangular HSS columns, in which the load transferred was predominantly shear. A review of costs also showed that single-plate and single-angle connections were the most economical, with double-angle and fillet-welded tee connections being more expensive. Through-plate and flare-bevel welded tee connections were among the most expensive (Sherman, 1995a). Over a wide range of connections tested, only one limit state was identified for the rectangular HSS column: punching shear failure related to end rotation of the beam, when a thick shear plate was joined to a relatively thin-walled HSS. Compliance with the inequality given by K1-10 precludes this HSS failure mode. This design rule is valid providing the HSS wall is not classified as a *slender element*. An extrapolation of inequality K1-10 has also been made for round HSS columns, subject to the round HSS cross-section not being classified as a *slender element*.

6. Concentrated Axial Force on the End of a Rectangular HSS with a Cap Plate

In Section K1.6, two limit states are given for the strength of a square or rectangular HSS wall with load transferred through a cap plate (or the flange of a T-stub), as shown in Figure C-K1.2. In general, the rectangular HSS could have dimensions of $B \times H$, but the illustration shows the bearing length (or width), N, oriented for lateral load dispersion into the wall of dimension B. A conservative distribution

TABLE C-K1.1 (a)
Nominal Strengths of Plate-to-Rectangular HSS Connections

Connection Type	Connection Nominal Strength
Longitudinal plate	$\beta \le 0.85$ Basis: chord wall plastification
	$R_n = \dfrac{F_y t^2}{1 - \dfrac{t_p}{B}} \left(\dfrac{2N}{B} + 4\sqrt{1 - \dfrac{t_p}{B} Q_f} \right)$
Transverse plate	$\beta \approx 1.0$ Basis: HSS side wall strength
	Tension and compression: $R_n = 2F_y t[5k + N]$ Compression in T-connections: $R_n = 1.6t^2 \left[1 + \dfrac{3N}{H - 3t} \right] \sqrt{EF_y}\, Q_f$ Compression in cross-connections: $R_n = \dfrac{48t^3}{H - 3t} \sqrt{EF_y}\, Q_f$
	$0.85 \le \beta \le 1 - 2t/B$ Basis: punching shear failure
where $\beta = \dfrac{B_p}{B}$	$R_n = 0.6F_y t[2t_p + 2B_{ep}]$
	All β Basis: uneven load distribution
	$R_n = \dfrac{10}{B/t} F_y t B_p \le F_{yp} t_p B_p$

Functions and Range of Validity

$\dfrac{B}{t} \le 35$ for the loaded HSS wall in transverse connections and

≤ 40 for longitudinal connections

$0.25 < \dfrac{B_p}{B} \le 1.0$ for transverse connections

$B_{ep} = \dfrac{10B_p}{B/t}$ but $\le B_p$

$k =$ outside corner radius of HSS $\ge 1.5t$

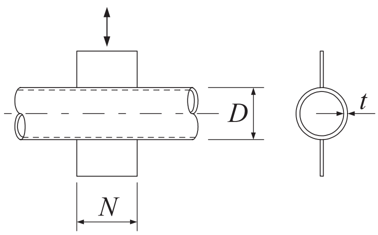
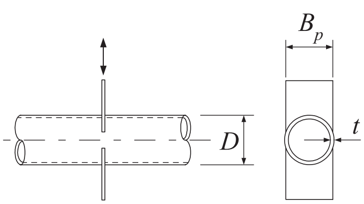
$Q_f = 1.0$ (chord in tension, for transverse connections)

$Q_f = 1.3 - 0.4\dfrac{U}{\beta}$ but ≤ 1.0 (chord in compression, for transverse connections)

$Q_f = \sqrt{1 - U^2}$ (for longitudinal connections)

slope can be assumed as 2.5:1 from each face of the tee web (Wardenier and others, 1991; Kitipornchai and Traves, 1989), which produces a dispersed load width of $(5t_p + N)$. If this is less than B, only the two side walls of dimension B are effective in resisting the load, and even they will both be only partially effective. If $(5t_p + N) \ge B$, all four walls of the rectangular HSS will be engaged, and all

TABLE C-K1.1 (b)
Nominal Strengths of Plate-to-Round HSS Connections

	Connection Nominal Strength		
Connection Type	**Axial Force**	**Bending in Plane**	**Bending out of Plane**
Longitudinal plate 	Chord plastification: $R_n = 5.5 F_y t^2 \left(1 + 0.25 \frac{N}{D}\right) Q_f$	$M_n = N R_n$	—
Transverse plate 	$R_n = F_y t^2 \left[\frac{5.5}{1 - 0.81 \frac{B_p}{D}}\right] Q_f$	—	$M_n = 0.5 B_p R_n$

Functions and Range of Validity

$\frac{D}{t} \leq 50$ for T-connections and ≤ 40 for cross-connections

$0.2 < \frac{B_p}{D} \leq 1.0$ for transverse connections

$Q_f = 1.0$ (chord in tension)

$Q_f = 1.0 - 0.3U(1 + U)$ but ≤ 1.0 (chord in compression)

will be fully effective; however, the cap plate (or T-stub flange) must be sufficiently thick for this to happen. In Equations K1-11 and K1-12 the size of any weld legs has been conservatively ignored. If the weld leg size is known, it is acceptable to assume load dispersion from the toes of the welds. The same load dispersion model as shown in Figure C-K1.2 can also be applied to round HSS-to-cap plate connections.

K2. HSS-TO-HSS TRUSS CONNECTIONS

The classification of HSS truss-type connections as K- (which includes N-), Y- (which includes T-), or cross- (also known as X-) connections is based on the method of force transfer in the connection, not on the physical appearance of the connection. Examples of such classification are shown in Figure C-K2.1.

As noted in Section K2, when branch members transmit part of their load as K-connections and part of their load as T-, Y-, or cross-connections, the adequacy of each branch is determined by linear interaction of the proportion of the branch load involved in each type of load transfer. One K-connection, shown in Figure C-K2.1(b), illustrates that the branch force components normal to the chord member may differ by as much as 20 percent and still be deemed to exhibit K-connection behavior. This is to accommodate slight variations in branch member forces along a typical truss, caused by a series of panel point loads. The N-connection in Figure C-K2.1(c), however, has a ratio of branch force components normal to the chord member of 2:1. In this case, the connection is analyzed as both a "pure" K-connection (with balanced branch forces) and a cross- (or X-) connection (because the remainder of the diagonal branch load is being transferred through the connection), as shown in Figure C-K2.2. For the diagonal tension branch in that connection, the following check is also made:

$$\begin{aligned}&(0.5P\sin\theta / K\text{-connection available strength})\\ &+ (0.5P\sin\theta/\text{cross-connection available strength}) \le 1.0\end{aligned}$$

If the gap size in a gapped K- (or N-) connection [for example, Figure C-K2.1(a)] becomes large and exceeds the value permitted by the eccentricity limit, the "K-connection" should be treated as two independent Y-connections. In cross-connections, such as Figure C-K2.1(e), where the branches are close together or overlapping, the combined "footprint" of the two branches can be taken as the loaded area on the chord member. In K-connections such as Figure C-K2.1(d), where a branch has very little or no loading, the connection can be treated as a Y-connection, as shown.

The design of welded HSS connections is based on potential limit states that may arise for a particular connection geometry and loading, which in turn represent possible failure modes that may occur within prescribed limits of applicability.

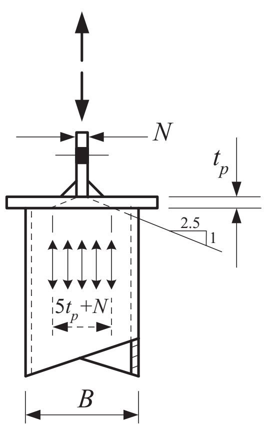


Fig. C-K1.2. Load dispersion from a concentrated force through a cap plate.

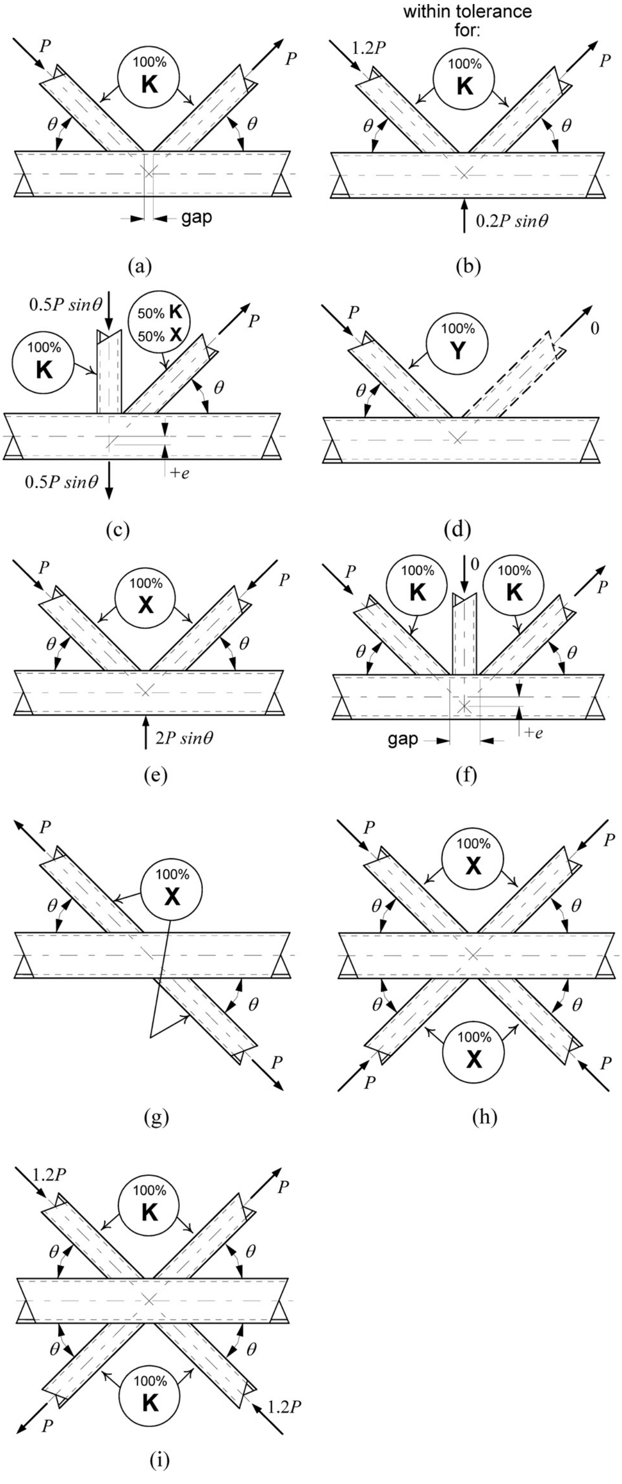


Fig. C-K2.1. Examples of HSS connection classification.

Some typical failure modes for truss-type connections, shown for rectangular HSS, are given in Figure C-K2.3.

1. Definitions of Parameters

Some parameters are defined in Figure C-K1.1.

2. Criteria for Round HSS

The limits of validity in Section K2.2a generally represent the parameter range over which the equations have been verified in experiments. The following limitations bear explanation:

(2) The minimum branch angle is a practical limit for good fabrication. Smaller branch angles are possible, but prior agreement with the fabricator should be made.
(5) The wall slenderness limit for the compression branch is a restriction so that connection strength is not reduced by branch local buckling.
(6) The minimum width ratio limit for gapped K-connections has been added in this Specification as a precaution, because Packer (2004) showed that for width ratios less than 0.4, Equation K2-6 may be potentially unconservative when evaluated against proposed equations for the design of such connections by the American Petroleum Institute (API, 1993).
(7) The restriction on the minimum gap size is only stated so that adequate space is available to enable welding at the toes of the branches to be satisfactorily performed.
(8) The restriction on the minimum overlap is applied so that there is an adequate interconnection of the branches, to enable effective shear transfer from one branch to the other.

The provisions given in Sections K2.2b and K2.2c are generally based, with the exception of the punching shear provision, on semi-empirical "characteristic strength" expressions, which have a confidence of 95 percent, taking into account the variation in experimental test results as well as typical variations in mechanical and geometric properties. These "characteristic strength" expressions are then multiplied by resistance factors for LRFD or divided by safety factors for ASD to further allow for the relevant failure mode. In the case of the chord plastification failure mode a ϕ factor of 0.9 or Ω factor of 1.67 is applied, whereas in the case

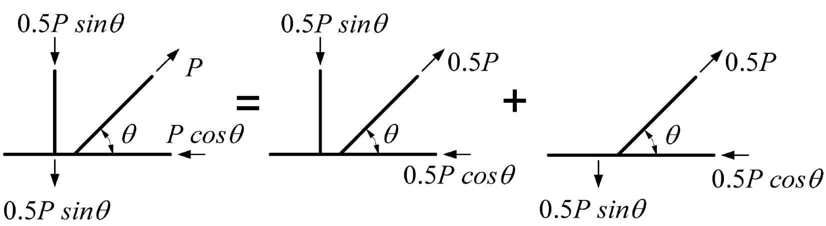
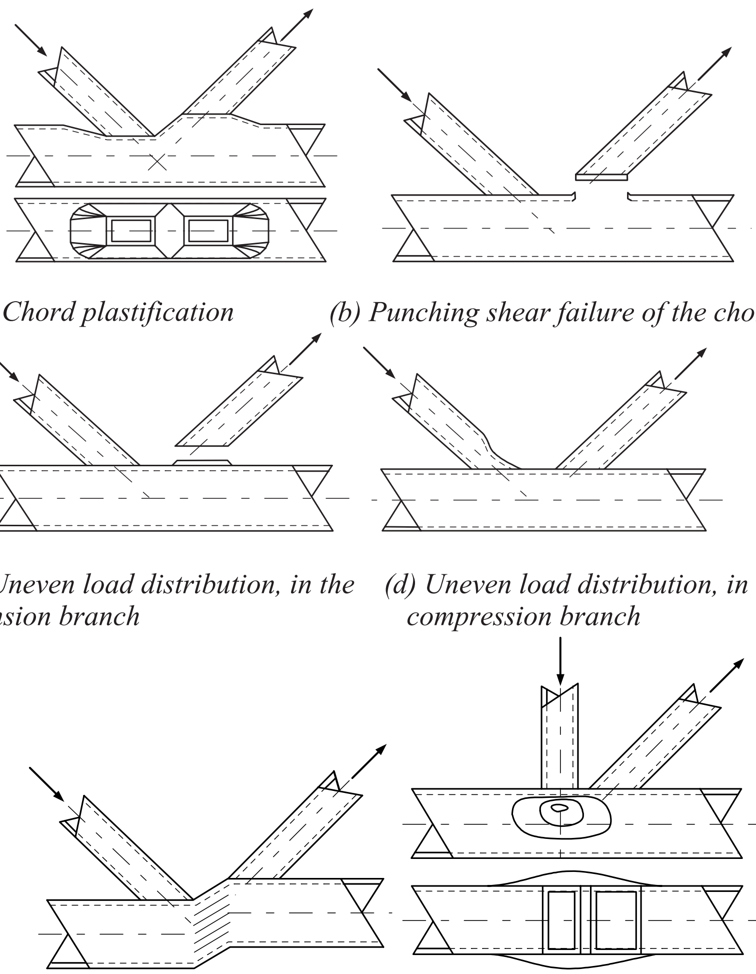


Fig. C-K2.2. Checking of K-connection with imbalanced branch member loads.

of punching shear a ϕ factor of 0.95 or a Ω factor of 1.58 is applied. The latter ϕ factor is 1.0 (equivalent to Ω of 1.50) in many recommendations or specifications [for example, IIW (1989), Packer and Henderson (1997), and Wardenier and others (1991)] to reflect the large degree of reserve strength beyond the analytical nominal strength expression, which is itself based on the shear yield (rather than ultimate) strength of the material. In this Specification, however, a ϕ factor of 0.95 or Ω factor of 1.58 is applied to maintain consistency with the factors for similar failure modes in Section K2.3. The shear failure resistance has also been taken as $0.95(0.6F_y) = 0.57F_y$, and elsewhere in Sections K2 and K3 as well, whereas



(a) Chord plastification

(b) Punching shear failure of the chord

(c) Uneven load distribution, in the tension branch

(d) Uneven load distribution, in the compression branch

(e) Shear yielding of the chord, in the gap

(f) Chord sidewall failure

Fig. C-K2.3. Typical limit states for HSS-to-HSS truss connections.

IIW (1989) uses a von Mises shear yield resistance formulation of $1.0(F_y/\sqrt{3}) = 0.58F_y$. One should note that if the ultimate stress, F_u, were adopted as a basis for a punching shear rupture criterion, the accompanying ϕ would be 0.75 and Ω would be 2.0, as elsewhere in this Specification. Then, $0.75(0.6\ F_u) = 0.45\ F_u$ would yield a very similar value to $0.95(0.6\ F_y) = 0.57\ F_y$, and in fact the latter is even more conservative for HSS with specified nominal F_y/F_u ratios less than 0.79. Equation K2-4 need not be checked when $\beta > (1 - 1/\gamma)$ because this is the physical limit at which the branch can punch into (or out of) the main tubular member.

With round HSS in axially loaded K-connections, the size of the compression branch dominates the determination of the connection strength. Hence, the term D_b in Equation K2-6 pertains only to the compression branch and is not an average of the two branches. Thus, if one requires the connection strength expressed as a force in the tension branch, one can resolve the answer from Equation K2-6 into the direction of the tension branch, using Equation K2-8. That is, it is not necessary to repeat a calculation similar to Equation K2-6 with D_b as the tension branch. Note that Section K2.2c deals with branches subject to axial loading only. This is because there should only be axial forces in the branches of a typical planar K-connection if the truss structural analysis is performed according to one of the recommended methods, which are:

(i) pin-jointed analysis; or
(ii) analysis using web members pin-connected to continuous chord members, as shown in Figure C-K2.4.

3. Criteria for Rectangular HSS

The limits of validity in Section K2.3a generally represent the parameter range over which the design provisions have been verified in experiments. They are also

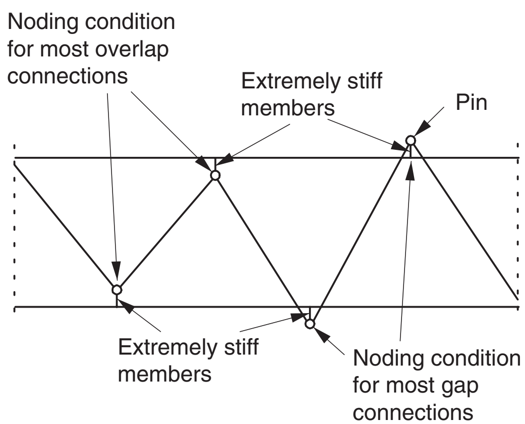


Fig. C-K2.4. Modeling assumption using web members pin-connected to continuous chord members.

set to eliminate the occurrence of certain failure modes for particular connection types, thereby making connection design easier. The following limitations from Section K2.3a bear explanation:

(2) The minimum branch angle is another practical limit for fabrication. Smaller branch angles are possible, but prior agreement with the fabricator should be made.

(8) The restriction on the minimum overlap is applied to ensure that there is an adequate interconnection of the branches to provide effective shear transfer from one branch to the other.

The restriction on the minimum gap ratio in Section K2.3c is modified from IIW (1989), according to Packer and Henderson (1997), to be more practical. The minimum gap size, g, is only specified so that adequate space is available to enable welding at the toes of the branches to be satisfactorily performed.

Equation K2-13 represents an analytical yield line solution for flexure of the connecting chord face. This nominal strength equation serves to limit connection deformations and is known to be well below the ultimate connection strength. A ϕ factor of 1.0 or Ω factor of 1.5 is thus appropriate. When the branch width exceeds 0.85 of the chord width this yield line failure mechanism will result in a noncritical design load.

The limit state of punching shear, evident in Equations K2-14 and K2-21, is based on the effective punching shear perimeter around the branch, with the total branch perimeter being an upper limit on this length. The term β_{eop} represents the chord face effective punching shear width ratio, adjacent to one (Equation K2-21) or two (Equation K2-14) branch walls transverse to the chord axis. This β_{eop} term incorporates a ϕ factor of 0.8 or Ω factor of 1.88. Applying to generally one dimension of the rectangular branch footprint, this was deemed by AWS to be similar to a global ϕ factor of 0.95 or Ω factor of 1.58 for the whole expression, so this expression for punching shear was implemented into AWS (2004) with an overall ϕ of 0.95. This ϕ factor of 0.95 or Ω factor of 1.58 has been carried over to this Specification and this topic is discussed further in Section K2.2. Notes below Equations K2-14 and K2-21 indicate when this failure mode is either physically impossible or noncritical. In particular, note that Equation K2-21 is noncritical for square HSS branches.

Equation K2-15 is generally in accord with a limit state given in IIW (1989), but with the k term [simply t in IIW (1989)] modified to be compatible with Equation K1-4, which in turn is derived from loads on I-shaped members. Equations K2-16 and K2-17 are in a format different than used internationally [for example, IIW (1989)] for this limit state and are unique to this Specification, having been replicated from Equations K1-5 and K1-6, along with their associated ϕ and Ω factors. These latter equations in turn are HSS versions (for two webs) of equations for I-shaped members with a single web.

The limit state of "uneven load distribution", which is manifested by local buckling of a compression branch or premature yield failure of a tension branch, represented by Equations K2-18 and K2-22, is checked by summing the effective areas of the four sides of the branch member. For T-, Y- and cross-connections the two walls of the branch transverse to the chord are likely to be only partially effective (Equation K2-18), whereas for gapped K-connections one wall of the branch transverse to the chord is likely to be only partially effective (Equation K-22). This reduced effectiveness is primarily a result of the flexibility of the connecting face of the chord, as incorporated in Equations K2-19 and K2-23. The effective width term b_{eoi} has been derived from research on transverse plate-to-HSS connections (as cited below for overlapped K-connections) and incorporates a ϕ factor of 0.8 or Ω factor of 1.88. Applying the same logic described above for the limit state of punching shear, a global ϕ factor of 0.95 or Ω factor of 1.58 was adopted in AWS D1.1 (AWS, 2004), and this has been carried over to this Specification [although, as noted previously, a ϕ factor of 1.0 is used in IIW (1989)].

For T-, Y- and cross-connections with $\beta \leq 0.85$, the connection strength is determined by Equation K2-13 only.

For axially loaded, gapped K-connections, plastification of the chord connecting face under the "push-pull" action of the branches is by far the most prevalent and critical failure mode. Indeed, if all the HSS members are square, this failure mode is critical and Equation K2-20 is the only one to be checked. This formula for chord face plastification is a semi-empirical "characteristic strength" expression, which has a confidence of 95 percent, taking into account the variation in experimental test results as well as typical variations in mechanical and geometric properties. Equation K2-20 is then multiplied by a ϕ factor for LRFD or divided by an Ω factor for ASD to further allow for the failure mode and provide an appropriate safety margin. A reliability calibration (Packer, Birkemoe, and Tucker, 1984) for this equation, using a database of 263 gapped K-connections and the exponential expression for the resistance factor (with a safety index of 3.0 and a coefficient of separation of 0.55) derived a ϕ factor of 0.89 (Ω factor of 1.69), while also imposing the parameter limits of validity. Since this failure mode dominates the test database, there is insufficient supporting test data to calibrate Equations K2-21 and K2-22.

For the limit state of shear yielding of the chord in the gap of gapped K-connections, Section K2.3c(c) differs from international practice [for example, IIW (1989)] but recommends application of another section of this Specification, Section G5. This limit state need only be checked if the chord member is rectangular (in other words, not square) and is also oriented such that the shorter wall of the chord section lies in the plane of the truss, hence providing a more critical chord shear condition due to the short "webs." The axial force present in the gap region of the chord member may also have an influence on the shear capacity of the chord webs in the gap region.

For K-connections, the scope covers both gapped and overlapped connections, although the latter are generally more difficult and more expensive to fabricate than

K-connections with a gap. However, an overlapped connection will, in general, produce a connection with a higher static strength, a stiffer truss, and a connection with a higher fatigue resistance, than its gapped connection counterpart. Note that Sections K2.3c and K2.3d deal with branches subject to axial loading only. This is because there should only be axial forces in the branches of a typical planar K-connection if the truss structural analysis is performed according to one of the recommended methods, which are:

(i) pin-jointed analysis, or
(ii) analysis using web members pin-connected to continuous chord members, as shown in Figure C-K2.4.

For rectangular HSS, the sole failure mode to be considered for design of overlapped connections is the limit state of "uneven load distribution" in the branches, manifested by either local buckling of the compression branch or premature yield failure of the tension branch. The design procedure presumes that one branch is welded solely to the chord and hence only has a single cut at its end. This can be considered "good practice" and the "thru member" is termed the overlapped member. For partial overlaps of less than 100 percent, the other branch is then double-cut at its end and welded to both the thru branch as well as the chord. The branch to be selected as the "thru" or overlapped member should be the one with the larger overall width. If both branches have the same width, the thicker branch should be the overlapped branch. For a single failure mode to be controlling (and not have failure by one branch punching into or pulling out of the other branch, for example), limits are placed on various connection parameters, including the relative width and relative thickness of the two branches. The foregoing fabrication advice for rectangular HSS also pertains to round HSS overlapped K-connections, but the latter involves more complicated profiling of the branch ends to provide good saddle fits.

Overlapped rectangular HSS K-connection strength calculations (Equations K2-24, K2-25 and K2-26) are performed initially just for the overlapping branch, regardless of whether it is in tension or compression, and then the resistance of the overlapped branch is determined from that. The equations for connection strength, expressed as a force in a branch, are based on the load-carrying contributions of the four side walls of the overlapping branch and follow the design recommendations of the International Institute of Welding (IIW, 1989; Packer and Henderson, 1997; AWS, 2004). The effective widths of overlapping branch member walls transverse to the chord (b_{eoi} and b_{eov}) depend on the flexibility of the surface on which they land, and are derived from plate-to-HSS effective width measurements (Rolloos, 1969; Wardenier, Davies, and Stolle, 1981; Davies and Packer, 1982). The constant of 10 in the b_{eoi} and b_{eov} terms has already been reduced from values determined in tests and incorporates a ϕ factor of 0.80 or Ω factor of 1.88 in those terms. Applying the same logic described above for the limit state of punching shear in T-, Y- and cross-connections, a global ϕ factor of 0.95 or Ω factor of 1.58 was adopted by AWS D1.1 and this has been carried over to this Specification [although as noted previously a ϕ factor of 1.0 is used by IIW (1989)].

The applicability of Equations K2-24, K2-25 and K2-26 depends on the amount of overlap, O_v, where $O_v = (q/p) \times 100\%$. It is important to note that p is the projected length (or imaginary footprint) of the overlapping branch on the connecting face of the chord, even though it does not physically contact the chord. Also, q is the overlap length measured along the connecting face of the chord beneath the region of overlap of the branches. This is illustrated in Figure C-K1.1.

A maximum overlap of 100 percent occurs when one branch sits completely on the other branch. In such cases, the overlapping branch is sometimes moved slightly up the overlapped branch so that the heel of the overlapping branch can be fillet welded to the face of the overlapped branch. If the connection is fabricated in this manner, an overlap slightly greater than 100 percent is created. In such cases, the connection strength for a rectangular HSS connection can be calculated by Equation K2-26 but with the B_{bi} term replaced by another b_{eov} term. Also, with regard to welding details, it has been found experimentally that it is permissible to just tack weld the "hidden toe" of the overlapped branch, providing that the components of the two branch member forces normal to the chord substantially balance each other. The "hidden toe" should be fully welded to the chord if the normal components of the two branch forces differ by more than 20 percent. If the components of the two branch forces normal to the chord do in fact differ significantly, the connection should also be checked for behavior as a T-, Y- or cross-connection, using the combined footprint and the net force normal to the chord (see Figure C-K2.1).

The design of "Welds to Branches" may be performed in either of two ways:

(a) The welds may be proportioned to develop the capacity of the connected branch wall, at all points along the weld length. This may be appropriate if the branch loading is complex or the loading is not known by the weld designer. Welds sized in this manner represent an upper limit on the required weld size and may be excessively conservative in some situations.

(b) The welds may be designed as "fit for purpose," to resist branch forces that are typically known in HSS truss-type connections. Many HSS truss web members have low axial loads, for a variety of possible reasons, and in such situations this weld design philosophy is ideal. However, the nonuniform loading of the weld perimeter due to the flexibility of the connecting HSS face must be taken into account by using weld effective lengths. Suitable effective lengths for various rectangular HSS connections subject to branch axial loading are given in Section K2.3e. These provisions are similar to those given in AWS (2004) and are based on full-scale HSS connection and truss tests that studied weld failures (Frater and Packer, 1992; 1992a; Packer and Cassidy, 1995). Adequate reliability is still obtained with the effective length expressions given if the directional strength increase allowed with fillet welds is used (Packer, 1995). Examples of weld joints in which weld effective lengths are less than 100 percent of the total weld length are shown in Figure C-K2.5. Most HSS trusses have the web members inclined to the chord at angles less than

50 degrees, in which cases the weld length around each branch perimeter in a K-connection will be 100 percent effective, as can be seen from Equation K2-31. Similar advice to that given in Section K2.3e is replicated in Section K1.3b for welds to transverse plates joined to rectangular HSS.

K3. HSS-TO-HSS MOMENT CONNECTIONS

Section K3 on HSS-to-HSS connections under moment loading is applicable to frames with PR or FR moment connections, such as Vierendeel girders. The provisions of Section K3 are not generally applicable to typical planar triangulated trusses (which are covered by Section K2), since the latter should be analyzed in a manner which results in no bending moments in the web members (see Commentary on Section K2). Thus, K-connections with moment loading on the branches are not covered by this Specification.

Available testing for HSS-to-HSS moment connections is much less extensive than that for axially-loaded T-, Y-, cross- and K-connections. Hence, the governing limit states to be checked for axially-loaded connections have been used as a basis for the possible limit states in moment-loaded connections. Thus, the design criteria for round HSS moment connections are based on the limit states of chord plastification and punching shear failure, with ϕ and Ω factors consistent with Section K2, while the design criteria for rectangular HSS moment connections are based on the limit states of plastification of the chord connecting face, chord side wall crushing, uneven load distribution and chord distortional failure, with ϕ and Ω factors consistent with Section K2. The "chord distortional failure" mode is applicable only to rectangular HSS T-connections with an out-of-plane bending moment on the branch. Rhomboidal distortion of the branch can be prevented by the use of

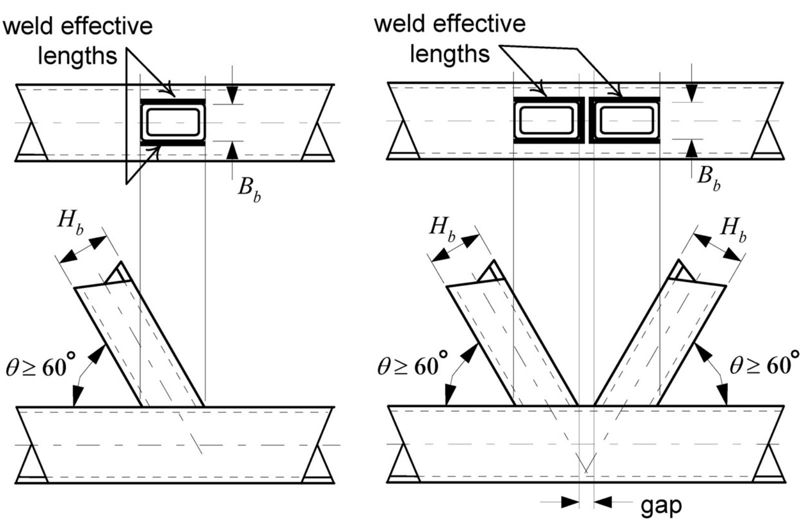


Fig. C-K2.5. Weld effective lengths for particular rectangular HSS connections.

stiffeners or diaphragms to maintain the rectangular cross-sectional shape of the chord. The limits of applicability of the equations in Section K3 are predominantly reproduced from Section K2. The basis for the equations in Section K3 is Eurocode 3 (2002), which represents one of the most up-to-date consensus specifications or recommendations on welded HSS-to-HSS connections. The equations in Section K3 have also been adopted in CIDECT Design Guide No. 9 (Kurobane and others, 2004).

CHAPTER L

DESIGN FOR SERVICEABILITY

L1. GENERAL PROVISIONS

Serviceability limit states are conditions in which the functions of a building are impaired because of local damage, deterioration or deformation of building components, or occupant discomfort. While serviceability limit states generally do not involve collapse of a building, loss of life or injury, they can seriously impair the building's usefulness and lead to costly repairs and other economic consequences. Serviceability provisions are essential to provide satisfactory performance of building structural systems. Neglect of serviceability may result in structures that are excessively flexible or otherwise perform unacceptably in service.

The three general types of structural behavior that are indicative of impaired serviceability in steel structures are:

(1) Excessive deflections or rotations that may affect the appearance, function or drainage of the building or may cause damaging transfer of load to nonstructural components and attachments;
(2) Excessive vibrations produced by the activities of the building occupants, mechanical equipment, or wind effects, which may cause occupant discomfort or malfunction of building service equipment; and
(3) Excessive local damage (local yielding, buckling, slip or cracking) or deterioration (weathering, corrosion and discoloration) during the service life of the structure.

Serviceability limit states depend on the occupancy or function of the building, the perceptions of its occupants, and the type of structural system. Limiting values of structural behavior intended to provide adequate levels of serviceability should be determined by a team consisting of the building owner/developer, the architect and the structural engineer after a careful analysis of all functional and economic requirements and constraints. In arriving at serviceability limits, the team should recognize that building occupants are able to perceive structural deformations, motions, cracking or other signs of distress at levels that are much lower than those that would indicate impending structural damage or failure. Such signs of distress may be viewed as an indication that the building is unsafe and diminish its economic value, and therefore must be considered at the time of design.

Service loads that may require consideration in checking serviceability include: (1) static loads from the occupants, snow or rain on the roof, or temperature fluctuations; and (2) dynamic loads from human activities, wind effects, the operation of mechanical or building service equipment, or traffic near the building. Service loads are loads that act on the structure at an arbitrary point in time, and may be only a fraction of the corresponding nominal load. The response of the structure to service loads generally can be analyzed assuming elastic behavior. Members that accumulate residual deformations under service loads also may require examination with respect to this long-term behavior.

Serviceability limit states and appropriate load combinations for checking conformance to serviceability requirements can be found in ASCE 7, *Minimum Design Loads for Buildings and Other Structures*, Appendix B, and the commentary to Appendix B (ASCE, 2002).

L2. CAMBER

Camber is frequently specified in order to provide a level surface under *permanent loads*, for reasons of appearance or for alignment with other work. In normal circumstances camber does nothing to prevent excessive deflection or vibration. Camber in trusses is normally created by adjustment of member lengths prior to making shop connections. It is normally introduced in beams by controlled heating of selected portions of the beam or by cold bending, or both. Designers should be aware of practical limits presented by normal fabricating and erection practices. The *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2005) provides tolerances on actual camber and recommends that all cambers be measured in the fabricating shop on unstressed members, along general guidelines. Further information on camber may be found in Ricker (1989).

L3. DEFLECTIONS

Excessive vertical deflections and misalignment arise primarily from three sources: (1) gravity loads, such as dead, live and snow loads; (2) effects of temperature, creep and differential settlement; and (3) construction tolerances and errors. Such deformations may be visually objectionable; cause separation, cracking or leakage of exterior cladding, doors, windows and seals; and cause damage to interior components and finishes. Appropriate limiting values of deformations depend on the type of structure, detailing and intended use (Galambos and Ellingwood, 1986). Historically, common deflection limits for horizontal members have been 1/360 of the span for floors subjected to reduced live load and 1/240 of the span for roof members. Deflections of about 1/300 of the span (for cantilevers, 1/150 of the length) are visible and may lead to general architectural damage or cladding leakage. Deflections greater than 1/200 of the span may impair operation of moveable components such as doors, windows and sliding partitions.

Deflection limits depend very much on the function of the structure and the nature of the supported construction. Traditional limits expressed as a fraction of the span length should not be extrapolated beyond experience. For example, the traditional limit of 1/360 of the span worked well for controlling cracks in plaster ceilings with spans common in the first half of the twentieth century. Many structures with more flexibility have performed satisfactorily with the now common, and more forgiving, ceiling systems. On the other hand, with the advent of longer structural spans, serviceability problems have been observed with flexible grid ceilings where actual deflections were far less than 1/360 of the span, because the distance between partitions or other elements that may interfere with ceiling deflection are far less than the span of the structural member. Proper control of deflections is a complex subject requiring careful application of professional judgment. West, Fisher, and Griffis (2003) provide an extensive discussion of the issues.

Deflection computations for composite beams should include an allowance for slip, creep and shrinkage (see Commentary Section I3.1).

In certain long-span floor systems, it may be necessary to place a limit (independent of span) on the maximum deflection to minimize the possibility of damage of adjacent nonstructural elements (ISO, 1977). For example, damage to nonload-bearing partitions may occur if vertical deflections exceed more than about $^3/_8$ in. (10 mm) unless special provision is made for differential movement (Cooney and King, 1988); however, many components can and do accept larger deformations.

Load combinations for checking static deflections can be developed using first-order reliability analysis (Galambos and Ellingwood, 1986). Current static deflection guidelines for floor and roof systems are adequate for limiting superficial damage in most buildings. A combined load with an annual probability of being exceeded of 5 percent is appropriate in most instances. For serviceability limit states involving visually objectionable deformations, repairable cracking or other damage to interior finishes, and other short-term effects, the suggested load combinations are:

$$D + L$$

$$D + 0.5S$$

For serviceability limit states involving creep, settlement or similar long-term or permanent effects, the suggested load combination is

$$D + 0.5L$$

The dead load effect, D, may be that portion of dead load that occurs following attachment of nonstructural elements. For example, in composite construction, the dead load effects frequently are taken as those imposed after the concrete has cured. For ceiling related calculations, the dead load effects may include only those loads placed after the ceiling structure is in place.

L4. DRIFT

Drift (lateral deflection) in a steel building is a serviceability issue primarily from the effects of wind. Drift limits are imposed on buildings to minimize damage to cladding and to nonstructural walls and partitions. Lateral frame deflection is evaluated for the building as a whole, where the applicable parameter is the *total building drift* (defined as the lateral frame deflection at the top of the most occupied floor divided by the height of the building to that level, Δ/H). For each floor, the applicable parameter is *interstory drift* [defined as the lateral deflection of a floor relative to the lateral deflection of the floor immediately below, divided by the distance between floors, $(\delta_n - \delta_{n-1})/h$].

Typical drift limits in common usage vary from $H/100$ to $H/600$ for *total building drift* and $h/200$ to $h/600$ for *interstory drift*, depending on building type and the type of cladding or partition materials used. The most widely used values are H (or h)/400 to H(or h)/500 (ASCE Task Committee on Drift Control of Steel Building Structures, 1988). An absolute limit on *interstory drift* is sometimes imposed by designers in light of evidence that damage to nonstructural partitions, cladding and glazing may occur if the *interstory drift* exceeds about 3/8 in. (10 mm), unless special detailing practices are employed to accommodate larger movements (Cooney and King, 1988; Freeman, 1977). Many components can accept deformations that are significantly larger. More specific information on the damage threshold for building materials is available in the literature (Griffis, 1993).

It is important to recognize that frame racking or shear distortion (in other words, strain) is the real cause of damage to building elements such as cladding and partitions. Lateral drift only captures the horizontal component of the racking and does not include potential vertical racking (as from differential column shortening in tall buildings), which also contributes to damage. Moreover, some lateral drift may be caused by rigid body rotation of the cladding or partition which by itself does not cause strain and therefore damage. A more precise parameter, the *drift damage index*, used to measure the potential damage, has been proposed (Griffis, 1993).

It must be emphasized that a reasonably accurate estimate of building drift is essential to controlling damage. The structural analysis must capture all significant components of potential frame deflection including flexural deformation of beams and columns, axial deformation of columns and braces, shear deformation of beams and columns, beam-column joint rotation (panel-zone deformation), the effect of member joint size, and the P-Δ effect (Charney, 1990). For many low rise steel frames with normal bay widths of 30 to 40 ft (9 to 12 m), use of center-to-center dimensions between columns without consideration of actual beam column joint size and panel zone effects will usually suffice for checking drift limits. The stiffening effect of nonstructural cladding, walls and partitions may be taken into account if substantiating information (stress versus strain behavior) regarding their effect is available.

The level of wind load used in drift limit checks varies among designers depending upon the frequency with which the potential damage can be tolerated. Some designers use the same nominal wind load (wind load specified by the building code without a load factor) as used for the strength design of the members (typically a 50 or 100 year mean recurrence interval wind load). Other designers use a 10 year or 20 year mean recurrence interval wind load (Griffis, 1993; ASCE, 2002). Use of factored wind loads (nominal wind load multiplied by the wind load factor) is generally considered to be very conservative when checking serviceability.

It is important to recognize that drift control limits by themselves in wind-sensitive buildings do not provide comfort of the occupants under wind load. See Section L6 for additional information regarding perception to motion in wind sensitive buildings.

L5. VIBRATION

The increasing use of high-strength materials with efficient structural systems and open plan architectural layouts leads to longer spans and more flexible floor systems having less damping. Therefore, floor vibrations have become an important design consideration. Acceleration is the recommended standard for evaluation.

An extensive treatment of vibration in steel-framed floor systems and pedestrian bridges is found in Murray and others (1997). This guide provides basic principles and simple analytical tools to evaluate steel-framed floor systems and footbridges for vibration serviceability due to human activities, including walking and rhythmic activities. Both human comfort and the need to control movement for sensitive equipment are considered.

L6. WIND-INDUCED MOTION

Designers of wind-sensitive buildings have long recognized the need for controlling annoying vibrations under the action of wind to protect the psychological well-being of the occupants (Chen and Robertson, 1972). The perception of building motion under the action of wind may be described by various physical quantities including maximum displacement, velocity, acceleration, and rate of change of acceleration (sometimes called “jerk”). Acceleration has become the standard for evaluation because it is readily measured in the field and can be easily calculated analytically. Human response to building motion is a complex phenomenon involving many psychological and physiological factors. Perception and tolerance thresholds of acceleration as a measure of building motion are known to depend on factors such as frequency of the building, occupant gender, age, body posture (sitting, standing or reclining), body orientation, expectation of motion, body movement, visual cues, acoustic clues, and the type of motion (translational or torsional) (ASCE, 1981). Different thresholds and tolerance levels exist for different people and responses can be very subjective. It is known that some people can become accustomed to building motion and tolerate higher levels than others. Limited research exists on this subject but certain standards have been applied for design as discussed below.

Acceleration in wind-sensitive buildings may be expressed as either root mean square (RMS) or peak acceleration. Both measures are used in practice and there is no clear agreement as to which is the more appropriate measure of motion perception. Some researchers believe that peak acceleration during wind storms is a better measure of actual perception but that RMS acceleration during the entire course of a wind storm is a better measure of actual discomfort. Target peak accelerations of 21 milli-g (0.021 times the acceleration of gravity) for commercial buildings (occupied mostly during daylight hours) and 15 milli-g for residential buildings (occupied during the entire day) under a 10-year mean recurrence interval wind storm have been successfully used in practice for many tall building designs (Griffis, 1993). The target is generally more strict for residential buildings because of the continuous occupancy, the perception that people are less sensitive and more tolerant at work than at home, the fact that there is more turnover in commercial buildings, and the fact that commercial buildings are more easily evacuated for peak wind events. Peak acceleration and RMS acceleration in wind sensitive buildings are related by the "peak factor" best determined in a wind tunnel study and generally in the range of 3.5 for tall buildings (in other words, peak acceleration = peak factor × RMS acceleration). Guidance for design acceleration levels used in building design may be found in the literature (Chen and Robertson, 1972; Griffis, 1993; Hansen and Reed, 1973; Irwin, 1986; NRCC, 1990).

It is important to recognize that perception to building motion is strongly influenced by building mass and available damping as well as stiffness (Vickery, Isyumov, and Davenport, 1983). For this reason, building drift limits by themselves should not be used as the sole measure of controlling building motion (Islam, Ellingwood, and Corotis, 1990). Damping levels for use in evaluating building motion under wind events are generally taken as approximately 1 percent of critical damping for steel buildings.

L7. EXPANSION AND CONTRACTION

The satisfactory accommodation of expansion and contraction cannot be reduced to a few simple rules, but must depend largely upon the judgment of a qualified engineer.

The problem is likely to be more serious in buildings with masonry walls than with prefabricated units. Complete separation of the framing at widely spaced expansion joints is generally more satisfactory than more frequently located devices that depend upon the sliding of parts in bearing, and usually less expensive than rocker or roller expansion bearings.

Creep and shrinkage of concrete and yielding of steel are among the causes, other than temperature, for dimensional changes. Conditions during construction, such as temperature effects before enclosure of the structure, should also be considered.

Guidelines for the recommended size and spacing of expansion joints in buildings may be found in NRC (1974).

L8. CONNECTION SLIP

In bolted connections with bolts in holes having only small clearances, such as standard holes and slotted holes loaded transversely to the axis of the slot, the amount of possible slip is small. Slip at these connections is not likely to have serviceability implications. Possible exceptions include certain unusual situations where the effect of slip is magnified by the configuration of the structure, such as a connection at the base of a shallow cantilever beam or post where a small amount of bolt slip may produce unacceptable rotation and deflection.

This Specification requires that connections with oversized holes or slotted holes loaded parallel to the axis of the slot be designed as slip-critical connections. For a discussion of slip at these connections see the Commentary to Section J3.8. Where slip at service loads is a realistic possibility in these connections, the effect of connection slip on the serviceability of the structure must be considered.

CHAPTER M

FABRICATION, ERECTION AND QUALITY CONTROL

M1. SHOP AND ERECTION DRAWINGS

Supplementary information relevant to shop drawing documentation and associated fabrication, erection and inspection practices may be found in the *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2005) and in Schuster (1997).

M2. FABRICATION

1. Cambering, Curving and Straightening

The use of heat for straightening or cambering members is permitted for A514/A514M and A852/A852M steel, as it is for other steels. However, the maximum temperature permitted is 1,100 °F (590 °C) compared to 1,200 °F (650 °C) for other steels.

Cambering of flexural members, when required by the contract documents, may be accomplished in various ways. In the case of trusses and girders, the desired curvature can be built in during assembly of the component parts. Within limits, rolled beams can be cold-cambered.

Local application of heat has long been used as a means of straightening or cambering beams and girders. The method depends upon an ultimate shortening of the heat-affected zones. A number of such zones, on the side of the member that would be subject to compression during cold-cambering or “gagging,” are heated enough to be “upset” by the restraint provided by surrounding unheated areas. Shortening takes place upon cooling.

While the final curvature or camber can be controlled by these methods, it must be realized that some deviation due to workmanship considerations and permanent change due to handling is inevitable. Camber is usually defined by one mid-ordinate, as control of more than one point is difficult and not normally required. Reverse cambers are difficult to achieve and are discouraged. Long cantilevers are sensitive to camber and may deserve closer control.

2. Thermal Cutting

Thermal cutting is preferably done by machine. The requirement for a positive preheat of 150 °F (66 °C) minimum when beam copes and weld access holes are thermally cut in ASTM A6/A6M hot-rolled shapes with a flange thickness exceeding 2 in. (50 mm) and in built-up shapes made of material more than 2 in.

(50 mm) thick tends to minimize the hard surface layer and the initiation of cracks. This requirement for preheat for thermal cutting does not apply when the radius portion of the access hole or cope is drilled and the thermally cut portion is essentially linear. Such thermally cut surfaces are required to be ground and inspected in accordance with Section J1.6.

4. Welded Construction

To avoid weld contamination, the light oil coating that is generally present after manufacturing an HSS should be removed with a suitable solvent in locations where welding will be performed. In cases where an external coating has been applied at the mill, the coating should be removed at the location of welding or the manufacturer should be consulted regarding the suitability of welding in the presence of the coating.

5. Bolted Construction

In most connections made with high-strength bolts, it is only required to install the bolts to the snug-tight condition. This includes bearing-type connections where slip is permitted and, for ASTM A325 or A325M bolts only, tension (or combined shear and tension) applications where loosening or fatigue due to vibration or load fluctuations are not design considerations.

It is suggested that snug-tight bearing-type connections with ASTM A325 or A490 bolts be used in applications where A307 bolts are permitted.

This section provides rules for the use of oversized and slotted holes paralleling the provisions that have been in the RCSC Specification since 1972 (RCSC, 2004), extended to include A307 bolts, which are outside the scope of the RCSC Specification.

The Specification previously limited the methods used to form holes, based on common practice and equipment capabilities. Fabrication methods have changed and will continue to do so. To reflect these changes, this Specification has been revised to define acceptable quality instead of specifying the method used to form the holes, and specifically to permit thermally cut holes. AWS C4.7, Sample 3, is useful as an indication of the thermally cut profile that is acceptable (AWS, 1977). The use of numerically controlled or mechanically guided equipment is anticipated for the forming of thermally cut holes. To the extent that the previous limits may have related to safe operation in the fabrication shop, fabricators are referred to equipment manufacturers for equipment and tool operating limits.

10. Drain Holes

Because the interior of an HSS is difficult to inspect, concern is sometimes expressed regarding internal corrosion. However, good design practice can eliminate the concern and the need for expensive protection.

Corrosion occurs in the presence of oxygen and water. In an enclosed building, it is improbable that there would be sufficient reintroduction of moisture to cause severe corrosion. Therefore, internal corrosion protection is a consideration only in HSS that are exposed to weather.

In a sealed HSS, internal corrosion cannot progress beyond the point where the oxygen or moisture necessary for chemical oxidation is consumed (AISI, 1970). The oxidation depth is insignificant when the corrosion process must stop, even when a corrosive atmosphere exists at the time of sealing. If fine openings exist at connections, moisture and air can enter the HSS through capillary action or by aspiration due to the partial vacuum that is created if the HSS is cooled rapidly (Blodgett, 1967). This can be prevented by providing pressure-equalizing holes in locations that make it impossible for water to flow into the HSS by gravity.

Situations where an internal protective coating may be required include: (1) open HSS where changes in the air volume by ventilation or direct flow of water is possible; and (2) open HSS subject to a temperature gradient that causes condensation. In such instances it may also be prudent to use a minimum $^5/_{16}$ in. (8 mm) wall thickness.

HSS that are filled or partially filled with concrete should not be sealed. In the event of fire, water in the concrete will vaporize and may create pressure sufficient to burst a sealed HSS. Care should be taken to ensure that water does not remain in the HSS during or after construction, since the expansion caused by freezing can create pressure that is sufficient to burst an HSS.

Galvanized HSS assemblies should not be completely sealed because rapid pressure changes during the galvanizing process tend to burst sealed assemblies.

11. Requirements for Galvanized Members

Cracking has been observed in steel members during hot-dip galvanizing. The occurrence of these cracks has been correlated to several characteristics including, but not limited to, highly restrained details, base material chemistry, galvanizing practices, and fabrication workmanship. The requirement to grind beam copes before galvanizing will not prevent all cope cracks from occurring during galvanizing. However, it has been shown to be an effective means to reduce the occurrence of this phenomenon.

Galvanizing of structural steel and hardware such as fasteners is a process that depends on special design detailing and fabrication to achieve the desired level of corrosion protection. ASTM publishes a number of standards relating to galvanized structural steel:

ASTM A123 (ASTM, 2002) provides a standard for the galvanized coating and its measurement and includes provisions for the materials and fabrication of the products to be galvanized.

ASTM A153 (ASTM, 2001) is a standard for galvanized hardware such as fasteners that are to be centrifuged.

ASTM A384 (ASTM, 2002a) is the Standard Practice for Safeguarding Against Warpage and Distortion During Hot-Dip Galvanizing. It includes information on factors that contribute to warpage and distortion as well as suggestions for correction for fabricated assemblies.

ASTM A385 (ASTM, 2001a) is the Standard Practice for Providing High Quality Zinc coatings. It includes information on base materials, venting, treatment of contacting surfaces, and cleaning. Many of these provisions should be indicated on design and detail drawings.

ASTM A780 (ASTM, 2001b) provides for repair of damaged and uncoated areas of hot-dip galvanized coatings.

M3. SHOP PAINTING

1. General Requirements

The surface condition of unpainted steel framing of long-standing buildings that have been demolished has been found to be unchanged from the time of its erection, except at isolated spots where leakage may have occurred. Even in the presence of leakage, the shop coat is of minor influence (Bigos, Smith, Ball, and Foehl, 1954).

This Specification does not define the type of paint to be used when a shop coat is required. Final exposure and individual preference with regard to finish paint are factors that determine the selection of a proper primer. A comprehensive treatment of the subject is found in SSPC (2000).

3. Contact Surfaces

Special concerns regarding contact surfaces of HSS should be considered. As a result of manufacturing, a light oil coating is generally present on the outer surface of the HSS. If paint is specified, HSS must be cleaned of this oil coating with a suitable solvent; see SSPC (2000).

5. Surfaces Adjacent to Field Welds

This Specification allows for welding through surface materials, including appropriate shop coatings that do not adversely affect weld quality nor create objectionable fumes.

M4. ERECTION

2. Bracing

For information on the design of temporary lateral support systems and components for low-rise buildings see Fisher and West (1997).

4. Fit of Column Compression Joints and Base Plates

Tests on spliced full-size columns with joints that had been intentionally milled out-of-square, relative to either strong or weak axis, demonstrated that the load-carrying capacity was the same as that for similar columns without splices (Popov and Stephen, 1977). In the tests, gaps of $^{1}/_{16}$ in. (2 mm) were not shimmed; gaps of $^{1}/_{4}$ in. (6 mm) were shimmed with nontapered mild steel shims. Minimum size partial-joint-penetration groove welds were used in all tests. No tests were performed on specimens with gaps greater than $^{1}/_{4}$ in. (6 mm).

5. Field Welding

The purpose of wire brushing shop paint on surfaces adjacent to joints to be field welded is to reduce the possibility of porosity and cracking and also to reduce any environmental hazard. Although there are limited tests that indicate that painted surfaces result in sound welds without wire brushing, other tests have resulted in excessive porosity and/or cracking when welding coated surfaces. Wire brushing to reduce the paint film thickness minimizes weld rejection. Grinding or other treatment beyond wire brushing is not necessary.

M5. QUALITY CONTROL

To facilitate quality control, inspection, and identification, reference should be made to the *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2005).

5. Identification of Steel

Material identification procedures should be sufficient to show the material specification designations and to tie the material to any special material requirements, such as notch toughness when specified.

APPENDIX 1

INELASTIC ANALYSIS AND DESIGN

1.1. GENERAL PROVISIONS

The design of statically indeterminate steel structures according to Appendix 1 is based on their *inelastic strength*. Although design could be performed according to Section B3.4 (ASD) if the appropriate load factor were included in the analysis, this process is more complicated than simply performing design according to Section B3.3 (LRFD). For this reason, only LRFD provisions are provided. An exception is permitted in Section 1.3, as discussed below.

1.2. MATERIALS

Extensive past research on the plastic and inelastic behavior of continuous beams, *rigid frames* and connections has amply demonstrated the suitability of steel with yield stress levels up to 65 ksi (450 MPa) (ASCE, 1971).

1.3. MOMENT REDISTRIBUTION

The provision of Section 1.3 has been a part of the Specification since the 1949 edition. The permission of applying a redistribution of 10 percent of the elastically calculated bending moment at points of interior support due to gravity loading on continuous compact beams gives partial recognition to the philosophy of plastic design. Figure C-A-1.1 illustrates the application of this provision by comparing calculated moment diagrams with the diagrams altered by this provision.

1.4. LOCAL BUCKLING

Inelastic design requires that, up to the formation of the plastic mechanism or up to the peak of the inelastic load-deflection curve, the moments at the plastic hinge locations remain at the level of the plastic moment. This implies that the member must have sufficient inelastic rotation capacity to permit the redistribution of the moments. Sections that are designated as compact in Section B4 have a rotation capacity of approximately 3 and are suitable for plastic design. The limiting width/thickness ratio designated as λ_r in Table B4.1 is the maximum slenderness ratio for this rotation capacity to be achieved. Further discussion of the antecedents of these provisions is given in Commentary Section B4.

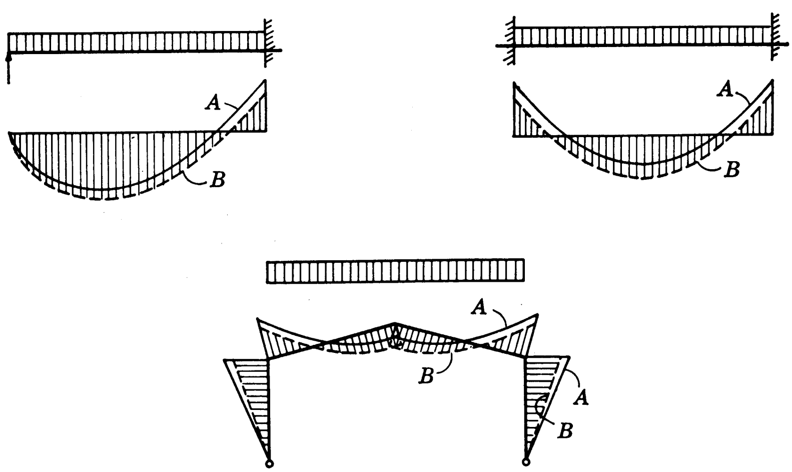
The additional slenderness limits in Equations A-1-1 through A-1-4 apply to cases not covered in Table B4.1. The equations for height-to-thickness ratio limits of webs of wide-flange members and rectangular HSS under combined flexure and compression have been taken from Table B5.1 of the 1999 *LRFD Specification* (AISC, 2000b). These provisions have been part of the plastic design requirements since the 1969 Specification, and they are based on research documented in *Plastic Design in Steel, A Guide and a Commentary* (ASCE, 1971). The equations for the flanges of HSS and other boxed sections (Equation A-1-3) and for round HSS (Equation A-1-4) are from the *Specification for the Design of Steel Hollow Structural Sections* (AISC, 2000).

The use of single-angle, tee and double-angle sections in statically indeterminate beams in plastic design is not recommended since the inelastic rotation capacity in the regions where the moment produces compression in an outstanding leg will typically not be sufficient.

1.5. STABILITY AND SECOND-ORDER EFFECTS

Section 1.5 requires that the equilibrium equations be formulated on the geometry of the deformed structure for frames designed by plastic or inelastic analysis.

Continuous, braced beams not subject to axial loads can be designed by *first-order plastic analysis*. Braced frames and moment frames having small axial loads in the members that are braced to prevent lateral-torsional buckling and loaded so as to produce bending about the major axis only may also be designed by first-order plastic analysis, provided that the requirements of Chapters C (the B_1 and B_2 amplification factors), E (column equations) and H (interaction equations) are accounted for. First-order plastic analysis is treated in ASCE (1971), in steel design textbooks [for example, Salmon and Johnson (1996) and Galambos, Lin, and Johnston (1996)], in textbooks dedicated entirely to plastic design [for example: Horne and Morris, (1982); Chen and Sohal (1995); and Bruneau, Uang, and



A = Actual moment diagram
B = Modified diagram corresponding to 10 percent moment reduction allowance at interior supports

Fig. C-A-1.1. Examples of effects of 10 percent moment redistribution.

Whittaker (1998)] and in structural engineering handbooks (Gaylord, Gaylord, and Stallmeyer, 1997).

First-order plastic analysis is applicable to continuous beams and low-rise frames with small axial loads. For these simple structures the tools of plastic analysis are readily available to the designer from books giving simple ways of calculating the plastic mechanism loads. This is not so for the case of general moment frames, where a full second-order inelastic analysis must be performed for the determination of the load effects on the members and the connections. The state-of-the-art of inelastic frame analysis is discussed in Chapter 16 of Galambos (1998). Textbooks [for example, Chen and Sohal (1995) and McGuire, Gallagher, and Ziemian (2000)] present the basic approaches to inelastic analysis, as well as worked examples and computer programs for use by students studying the subject.

1. Braced Frames

In Section 1.5.1 two constraints are given for the plastic design of braced frames: (1) the bracing system shall remain elastic; and (2) the axial force in any column must not exceed 85 percent of the *squash load*, $F_y A_g$.

2. Moment Frames

The provision in Section 1.5.2 restricts the axial force in any column to 75 percent of the *squash load*. This provision, as well as the corresponding one in Section 1.5.1, is a cautionary limitation because at high levels of axial force insufficient research has been conducted to ensure that sufficient inelastic rotation capacity remains in the member.

1.6. COLUMNS AND OTHER COMPRESSION MEMBERS

Columns in braced frames and moment frames that are designed on the basis of first-order inelastic analysis or a plastic mechanism analysis are proportioned according to the requirements of Section E3, with an effective length determined by methods of stability analysis. For moment frames, the effective length may exceed unity.

1.7. BEAMS AND OTHER FLEXURAL MEMBERS

The plastic moment, M_p, is the maximum moment that acts at the plastic hinge. When a wide-flange member is subject to flexure about its major axis, the ratio of the plastic moment to the yield moment is approximately 1.1 to 1.2. However, if flexure is about the minor axis, this ratio can exceed 1.6. A limit of $1.6M_y$ is imposed in order to prevent excessive yielding under service loads.

Portions of members that would be required to rotate inelastically as a plastic hinge, while the moments are redistributed to eventually form a plastic mechanism, need more closely spaced bracing than similar parts of a continuous frame designed in accordance with elastic theory. Equations A-1-7 and A-1-8 define the maximum permitted unbraced length in the vicinity of plastic hinges for wide-flange shapes bent about their major axis, and for rectangular shapes and symmetric box beams, respectively. These equations are identical to those in the 1999 *LRFD Specification* (AISC, 1999). They are different from the corresponding equations in Chapter N of the 1989 *ASD Specification* (AISC, 1989). The new equations are based on research reported in Yura and others (1978).

Some requirements that were in the plastic design chapter of the 1989 *ASD Specification* (AISC, 1989) are no longer explicitly enumerated in Appendix 1. One of these is the provision that web stiffeners are required at a point of load application where a plastic hinge would form. However, the provisions of Section J10 apply for plastic as well as elastic design. No mention is made of shear requirements, but the requirements of Chapter G apply. The plastic shear strength is $V_p = V_n = 0.6F_yA_w$ (Equation G2-1, with C_v equal to 1.0). The maximum permitted plastic web slenderness limit for plastic design is thus equal to

$$(h/t_w)_p = 1.1\sqrt{k_vE/F_y} = 1.1\sqrt{5E/F_y} = 2.5\sqrt{E/F_y} \qquad \text{(C-A-1-1)}$$

with a shear buckling coefficient $k_v = 5$. The plastic shear strength of $0.6F_yA_w$ is a liberalization of the previously used $0.55F_yA_w$ that was recommended in ASCE (1971) based on extensive research.

1.8. MEMBERS UNDER COMBINED FORCES

Members subject to bending moment and axial force are subject to the provisions of the interaction equations in Section H1. If the member contains a plastic hinge within its span or at its end, and bending is about the major axis of a doubly symmetric section, then the member must be laterally braced near the hinge location (Equation A-1-7 or A-1-8). When the unbraced length of the member exceeds these limits, the inelastic rotation capacity may be impaired, due to the combined influence of lateral and torsional deformation, to such an extent that plastic action is not achievable. However, if the required moment is small enough so the limitations of the interaction equations in Section H1 are fulfilled, the member will be strong enough to function at a joint where required hinge action is provided in another member entering the joint. If the forces on the beam-column include torsion, plastic design is not permitted by this Specification.

1.9. CONNECTIONS

The connections adjacent to plastic hinges must be designed with sufficient strength and ductility to sustain the forces and deformations imposed under the required loads. The practical implementation of this rule is that the applicable requirements of Chapter J must be strictly adhered to. The provisions for connection design in Chapter J have been developed from plasticity theory and verified by extensive testing, as discussed in ASCE (1971) and in many books and papers. Thus the connections that meet these provisions are inherently qualified for use in plastically designed structures.

APPENDIX 2

DESIGN FOR PONDING

Ponding stability is determined by ascertaining that the conditions of Equations A-2-1 and A-2-2 of Appendix 2 are fulfilled. These equations provide a conservative evaluation of the stiffness required to avoid runaway deflection, giving a factor of safety of four against ponding instability.

Since Equations A-2-1 and A-2-2 yield conservative results, it may be advantageous to perform a more detailed stress analysis to check whether a roof system that does not meet the above equations is still safe against ponding failure.

For the purposes of Appendix 2, *secondary members* are the beams or joists that directly support the distributed ponding loads on the roof of the structure, and *primary members* are the beams or girders that support the concentrated reactions from the *secondary members* framing into them. Representing the deflected shape of the primary and critical *secondary member* as a half-sine wave, the weight and distribution of the ponded water can be estimated, and, from this, the contribution that the deflection each of these members makes to the total ponding deflection can be expressed as follows (Marino, 1966):

For the *primary member*

$$\Delta_w = \frac{\alpha_p \Delta_o \left[1 + 0.25\pi\alpha_s + 0.25\pi\rho(1 + \alpha_s)\right]}{1 - 0.25\pi\alpha_p\alpha_s} \tag{C-A-2-1}$$

For the *secondary member*

$$\delta_w = \frac{\alpha_s \delta_o \left[1 + \dfrac{\pi^2}{32}\alpha_p + \dfrac{\pi^2}{8\rho}(1 + \alpha_p) + 0.185\alpha_s\alpha_p\right]}{1 - 0.25\pi\alpha_p\alpha_s} \tag{C-A-2-2}$$

In these expressions Δ_o and δ_o are, respectively, the primary and secondary beam deflections due to loading present at the initiation of ponding, and

$$\alpha_p = C_p / \left(1 - C_p\right)$$

$$\alpha_s = C_s / (1 - C_s)$$

$$\rho = \delta_o / \Delta_o = C_s / C_p$$

Using the above expressions for Δ_w and δ_w, the ratios Δ_w / Δ_o and δ_w / δ_o can be computed for any given combination of primary and secondary beam framing using the computed values of parameters C_p and C_s, respectively, defined in the Specification.

Even on the basis of unlimited elastic behavior, it is seen that the ponding deflections would become infinitely large unless

$$\left(\frac{C_p}{1-C_p}\right)\left(\frac{C_s}{1-C_s}\right) < \frac{4}{\pi} \qquad \text{(C-A-2-3)}$$

Since elastic behavior is not unlimited, the effective bending strength available in each member to resist the stress caused by ponding action is restricted to the difference between the yield stress of the member and the stress f_o produced by the total load supported by it before consideration of ponding is included.

Note that elastic deflection is directly proportional to stress. The admissible amount of ponding in either the primary or critical (midspan) *secondary member*, in terms of the applicable ratio Δ_w/Δ_o and δ_w/δ_o, can be represented as $(0.8F_y - f_o)/f_o$, assuming a factor of safety of 1.25 against yielding under the ponding load. Substituting this expression for Δ_w/Δ_o and δ_w/δ_o, and combining with the foregoing expressions for Δ_w and δ_w, the relationship between the critical values for C_p and C_s and the available elastic bending strength to resist ponding is obtained. The curves presented in Figures A-2.1 and A-2.2 are based upon this relationship. They constitute a design aid for use when a more exact determination of required flat roof framing stiffness is needed than given by the Specification provision that $C_p + 0.9C_s \leq 0.25$.

Given any combination of primary and secondary framing, the stress index is computed as follows:

For the *primary member*

$$U_p = \left(\frac{0.8F_y - f_o}{f_o}\right)_p \qquad \text{(C-A-2-4)}$$

For the *secondary member*

$$U_p = \left(\frac{0.8F_y - f_o}{f_o}\right)_s \qquad \text{(C-A-2-5)}$$

where

f_o = the stress due to $D + R$ (D = nominal dead load, R = nominal load due to rainwater or ice exclusive of the ponding contribution), ksi (MPa)

Depending upon geographic location, this loading should include such amount of snow as might also be present, although ponding failures have occurred more frequently during torrential summer rains when the rate of precipitation exceeded the rate of drainage runoff and the resulting hydraulic gradient over large roof areas caused substantial accumulation of water some distance from the eaves.

Given the size, spacing, and span of a tentatively selected combination of primary and secondary beams, for example, one may enter Figure A-2.1 at the level of the computed stress index U_p, determined for the primary beam; move horizontally to the computed C_s value of the secondary beams; then move downward to the abscissa scale. The combined stiffness of the primary and secondary framing is sufficient to prevent ponding if the flexibility constant read from this latter scale is larger than the value of C_p computed for the given *primary member*; if not, a stiffer primary or secondary beam, or combination of both, is required.

If the roof framing consists of a series of equally spaced wall-bearing beams, the beams would be considered as *secondary members*, supported on an infinitely stiff *primary member*. For this case, one would use Figure A-2.2. The limiting value of C_s would be determined by the intercept of a horizontal line representing the U_s value and the curve for $C_p = 0$.

The ponding deflection contributed by a metal deck is usually such a small part of the total ponding deflection of a roof panel that it is sufficient merely to limit its moment of inertia [in.^{4} per foot (mm^4 per meter) of width normal to its span] to 0.000025 (3940) times the fourth power of its span length, as provided in Equation A-2-2. However, the stability against ponding of a roof consisting of a metal roof deck of relatively slender depth-span ratio, spanning between beams supported directly on columns, may need to be checked. This can be done using Figures A-2.1 or A-2.2 with the following computed values:

U_p = stress index for the supporting beam
U_s = stress index for the roof deck
C_p = flexibility constant for the supporting beams
C_s = flexibility constant for one foot width of the roof deck ($S = 1.0$)

Since the shear rigidity of the web system is less than that of a solid plate, the moment of inertia of steel joists and trusses should be taken as somewhat less than that of their chords (Heinzerling, 1987).

APPENDIX 3

DESIGN FOR FATIGUE

When the limit state of fatigue is a design consideration, its severity is most significantly affected by the number of load applications, the magnitude of the stress range, and the severity of the stress concentrations associated with particular details. Issues of fatigue are not normally encountered in building design; however, when encountered and if the severity is great enough, fatigue is of concern and all provisions of Appendix 3 must be satisfied.

3.1. GENERAL

In general, members or connections subject to less than a few thousand cycles of loading will not constitute a fatigue condition except possibly for cases involving full reversal of loading and particularly sensitive categories of details. This is because the applicable cyclic design stress range will be limited by the static design stress. At low levels of cyclic tensile stress, a point is reached where the stress range is so low that fatigue cracking will not initiate regardless of the number of cycles of loading. This level of stress is defined as the *fatigue threshold*, F_{TH}.

Extensive test programs using full-size specimens, substantiated by theoretical stress analysis, have confirmed the following general conclusions (Fisher, Frank, Hirt, and McNamee, 1970; Fisher, Albrecht, Yen, Klingerman, and McNamee, 1974):

(1) Stress range and notch severity are the dominant stress variables for welded details and beams;
(2) Other variables such as minimum stress, mean stress, and maximum stress are not significant for design purposes; and
(3) Structural steels with yield points of 36 to 100 ksi (250 to 690 MPa) do not exhibit significantly different fatigue strengths for given welded details fabricated in the same manner.

3.2. CALCULATION OF MAXIMUM STRESSES AND STRESS RANGES

Fluctuation in stress that does not involve tensile stress does not cause crack propagation and is not considered to be a fatigue situation. On the other hand, in elements of members subject solely to calculated compressive stress, fatigue cracks may initiate in regions of high tensile *residual stress*. In such situations, the cracks generally do not propagate beyond the region of the residual tensile stress, because the *residual stress* is relieved by the crack. For this reason, stress ranges that are completely in compression need not be investigated for fatigue. For cases involving cyclic reversal of stress, the calculated stress range must be taken as the sum of the compressive stress and the tensile stress caused by different directions or patterns of the applied live load.

3.3. DESIGN STRESS RANGE

Fatigue resistance has been derived from an exponential relationship between the number of cycles to failure N and the stress range, S_r, called an $S - N$ relationship, of the form

$$N = \frac{C_f}{S_r^n} \qquad \text{(C-A-3-1)}$$

The general relationship is often plotted as a linear log-log function ($Log\ N = A - n\ Log\ S_r$). Figure C-A-3.1 shows the family of fatigue resistance curves identified as Categories A, B, B′, C, C′, D, E and E′. These relationships were established based on an extensive database developed in the United States and abroad (Keating and Fisher, 1986). The design stress range has been developed by adjusting the coefficient C_f so that a design curve is provided that lies two standard deviations of the standard error of estimate of the fatigue cycle life below the mean $S - N$ relationship of the actual test data. These values of C_f correspond to a probability of failure of 2.5 percent of the design life.

Prior to the 1999 AISC *Load and Resistance Factor Design Specification for Structural Steel Buildings* (AISC, 2000b), stepwise tables meeting the above criteria of cycles of loading, stress categories and design stress ranges were provided in the specifications. A single table format (Table A-3.1) was introduced in the 1999 AISC *LRFD Specification* that provides the stress categories, ingredients

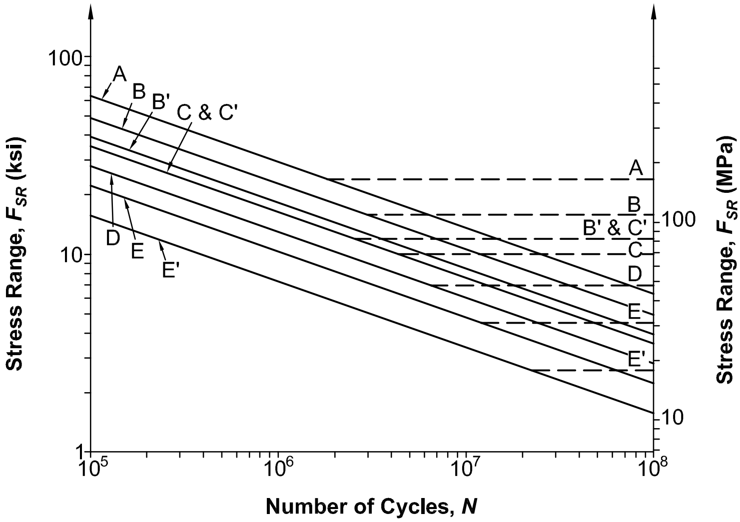


Fig. C-A-3.1. Fatigue resistance curves.

for the applicable equation, and information and examples including the sites of concern for potential crack initiation (AISC, 2000b).

Table A-3.1 is organized into 8 sections of general conditions for fatigue design, as follows:

– Section 1 provides information and examples for the steel material at copes, holes, cutouts or as produced.
– Section 2 provides information and examples for various types of mechanically fastened joints including eyebars and pin plates.
– Section 3 provides information related to welded connections used to join built-up members, such as longitudinal welds, access holes and reinforcements.
– Section 4 deals only with longitudinal load carrying fillet welds at shear splices.
– Section 5 provides information for various types of groove and fillet welded joints that are transverse to the applied cyclic stress.
– Section 6 provides information on a variety of groove welded attachments to flange tips and web plates as well as similar attachments connected with either fillet or partial-joint-penetration groove welds.
– Section 7 provides information on several short attachments to structural members.
– Section 8 collects several miscellaneous details such as shear connectors, shear on the throat of fillet, plug and slot welds, and their impact on base metal. It also provides for tension on the stress area of various bolts, threaded anchor rods and hangers.

A similar format and consistent criteria are used by other specifications.

When fabrication details involving more than one stress category occur at the same location in a member, the stress range at that location must be limited to that of the most restrictive category. The need for a member larger than required by static loading will often be eliminated by locating notch-producing fabrication details in regions subject to smaller ranges of stress.

A detail not explicitly covered before 1989 was added in 1999 to cover tension-loaded plate elements connected at their end by transverse partial-joint-penetration groove or fillet welds in which there is more than a single site for the initiation of fatigue cracking, one of which will be more critical than the others depending upon welded joint type and size and material thickness (Frank and Fisher, 1979). Regardless of the site within the joint at which potential crack initiation is considered, the design stress range provided is applicable to connected material at the toe of the weld.

3.4. BOLTS AND THREADED PARTS

The fatigue resistance of bolts subject to tension is predictable in the absence of pretension and prying action; provisions are given for such nonpretensioned details as hanger rods and anchor rods. In the case of pretensioned bolts, deformation of the connected parts through which pretension is applied introduces prying action, the magnitude of which is not completely predictable (Kulak and others, 1987). The effect of prying is not limited to a change in the average axial tension on the bolt but includes bending in the threaded area under the nut. Because of the uncertainties in calculating prying effects, definitive provisions for the design stress range for bolts subject to applied axial tension are not included in this Specification. To limit the uncertainties regarding prying action on the fatigue of pretensioned bolts in details which introduce prying, the design stress range provided in Table A-3.1 is appropriate for extended cyclic loading only if the prying induced by the applied load is small.

Nonpretensioned fasteners are not permitted under this Specification for joints subject to cyclic shear forces. Bolts installed in joints meeting all the requirements for slip-critical connections survive unharmed when subject to cyclic shear stresses sufficient to fracture the connected parts; provisions for such bolts are given in Section 2 of Table A-3.1.

3.5. SPECIAL FABRICATION AND ERECTION REQUIREMENTS

It is essential that when longitudinal backing bars are to be left in place, they be continuous or spliced using flush-ground complete-joint-penetration groove welds before attachment to the parts being joined. Otherwise, the transverse nonfused section constitutes a crack-like defect that can lead to premature fatigue failure or even *brittle fracture* of the built-up member.

In transverse joints subjected to tension a lack-of-fusion plane in T-joints acts as an initial crack-like condition. In groove welds, the root at the backing bar often has discontinuities that can reduce the fatigue resistance of the connection. Removing the backing, back gouging the joint, and rewelding eliminates the undesirable discontinuities.

The addition of contoured fillet welds at transverse complete-joint-penetration groove welds in T- and corner joints and at reentrant corners reduces the stress concentration and improves fatigue resistance.

Experimental studies on welded built-up beams demonstrated that if the surface roughness of flame-cut edges was less than 1000 μin. (25 μm), fatigue cracks would not develop from the flame-cut edge but from the longitudinal fillet welds connecting the beam flanges to the web (Fisher and others, 1970; Fisher and others, 1974). This provides Category B fatigue resistance without the necessity for grinding flame-cut edges.

Reentrant corners at cuts, copes and weld access holes provide a stress concentration point that can reduce fatigue resistance if discontinuities are introduced by punching or thermal cutting. Reaming sub-punched holes and grinding the thermally cut surface to bright metal prevents any significant reduction in fatigue resistance.

The use of run-off tabs at transverse butt-joint groove welds enhances weld soundness at the ends of the joint. Subsequent removal of the tabs and grinding of the ends flush with the edge of the member removes discontinuities that are detrimental to fatigue resistance.

APPENDIX 4

STRUCTURAL DESIGN FOR FIRE CONDITIONS

4.1. GENERAL PROVISIONS

Appendix 4 provides structural engineers with guidance in designing steel-framed building systems and components, including columns, and floor and truss assemblies, for fire conditions. Compliance with the performance objective in Section 4.1.1 can be demonstrated by either structural analysis or component qualification testing.

Thermal expansion and progressive decrease in strength and stiffness are the primary structural responses to elevated temperatures that may occur during fires. An assessment of a design of building components and systems based on structural mechanics that allows designers to address the fire-induced restrained thermal expansions, deformations and material degradation at elevated temperatures can lead to a more robust structural design for fire conditions.

Glossary

Terms pertinent to the design of structural components and systems for fire conditions are presented in the glossary. Terms in common with those in other fire-resistant design documents developed by the SFPE, ICC, NFPA, ASTM and similar organizations are defined in a manner consistent with those documents.

4.1.1. Performance Objective

The performance objective underlying the provisions and guidelines in this Specification is that of life safety. Fire safety levels should depend on the building occupancy, height of building, the presence of active fire mitigation measures, and the effectiveness of fire-fighting. Three limit states exist for elements serving as fire barriers (compartment walls and floors): (1) heat transmission leading to unacceptable rise of temperature on the unexposed surface; (2) breach of barrier due to cracking or loss of integrity; and (3) loss of load-bearing capacity. In general, all three must be considered by the engineer to achieve the desired performance. These three limit states are interrelated in fire-resistant design. For structural elements that are not part of a separating element, the governing limit state is loss of load-bearing capacity.

Specific performance objectives for a facility are determined by the stakeholders in the building process, within the context of the above general performance objective and limit states. In some instances, applicable building codes may stipulate that steel in buildings of certain occupancies and heights be protected by fire-resistant materials or assemblies to achieve specified performance goals.

4.1.4. Load Combinations and Required Strength

Fire safety measures are aimed at three levels: (1) to prevent the outbreak of fires through elimination of ignition sources or hazardous practices; (2) to prevent uncontrolled fire development and flashover through early detection and suppression; and (3) to prevent loss of life or structural collapse through fire protection systems, compartmentation, exit ways, and provision of general structural integrity and other passive measures. Specific structural design provisions to check structural integrity and risk of progressive failure due to severe fires can be developed from principles of structural reliability theory (Ellingwood and Leyendecker, 1978; Ellingwood and Corotis, 1991).

The limit state probability of failure due to fire can be written as

$$P(F) = P(F|D,I)P(D|I)P(I) \qquad \text{(C-A-4-1-1)}$$

where $P[I]$ = probability of ignition, $P[D|I]$ = probability of development of a structurally significant fire, and $P[F|D,I]$ = probability of failure, given the occurrence of the two preceding events. Measures taken to reduce $P(I)$and $P(D|I)$ are mainly nonstructural in nature. Measures taken by the structural engineer to design fire resistance into the structure impact $P(F|D,I)$.

The development of structural design requirements requires a target reliability level, reliability being measured by $P(F)$ in Equation C-A-4-1-1. Analysis of reliability of structural systems for gravity dead and live load (Galambos, Ellingwood, MacGregor, and Cornell, 1982) suggests that the limit state probability of individual steel members and connections is on the order of 10^{-5} to 10^{-4}/year. For redundant steel frame systems, $P(F)$ is on the order of 10^{-6} to 10^{-5}. The *de minimis* risk, that is, the level below which the risk is of regulatory or legal concern and the economic or social benefits of risk reduction are small, is on the order of 10^{-7} to 10^{-6}/year (Pate-Cornell, 1994). If $P(I)$ is on the order of 10^{-4}/year for typical buildings and $P(D|I)$ is on the order of 10^{-2} for office or commercial buildings in urban areas with suppression systems or other protective measures, then $P(F|D,I)$ should be approximately 0.1 to ascertain that the risk due to structural failure caused by fire is socially acceptable.

The use of first-order structural reliability analysis based on this target (conditional) limit state probability leads to the gravity load combination presented as Equation A-4-1. Load combination Equation A-4-1 is the same as Equation C2-3 that appears in Commentary C2.5 of SEI/ASCE 7 (ASCE, 2002), where the probabilistic bases for load combinations for extraordinary events is explained in detail. The factor 0.9 is applied to the dead load when the effect of the dead load is to stabilize the structure; otherwise, the factor 1.2 is applied. The companion action load factors on L and S in that equation reflect the fact that the probability of a coincidence of the peak time-varying load with the occurrence of a fire is negligible (Ellingwood and Corotis, 1991).

Commentary C2.5 of ASCE (2002) contains a second equation that includes 0.2W. That equation is provided so that the stability of the system is checked. The same purpose is accomplished by requiring that the frame be checked under the effect of a small notional lateral load equal to 0.2 percent of story gravity force, acting in combination with the gravity loads. The required strength of the structural component or system designed using these load combinations is on the order of 60 percent to 70 percent of the required strength under full gravity or wind load at normal temperature.

4.2. STRUCTURAL DESIGN FOR FIRE CONDITIONS BY ANALYSIS

4.2.1. Design-Basis Fire

Once a fuel load has been agreed upon for the occupancy, the designer should demonstrate the effect of various fires on the structure by assessing the temperature-time relationships for various ventilation factors. These relations may result in different structural responses, and it is useful to demonstrate the capability of the structure to withstand such exposures. The effects of a localized fire should also be assessed to ascertain that local damage is not excessive. Based on these results, connections and edge details can be specified to provide a structure that is sufficiently robust.

4.2.1.1. Localized Fire

Localized fires may occur in large open spaces, such as the pedestrian area of covered malls, concourses of airport terminals, warehouses, and factories, where fuel packages are separated by large aisles or open spaces. In such cases, the radiant heat flux can be estimated by a point source approximation, requiring the heat release rate of the fire and separation distance between the center of the fuel package and the closest surface of the steelwork. The heat release rate can be determined from experimental results or may be estimated if the mass loss rate per unit floor area occupied by the fuel is known. Otherwise, a steady-state fire may be assumed.

4.2.1.2. Post-Flashover Compartment Fires

Caution should be exercised when determining temperature-time profiles for spaces with high aspect ratios, for example, 5:1 or greater, or for large spaces, for example, those with an open (or exposed) floor area in excess of 5,000 ft^2 (465 m^2). In such cases, it is unlikely that all combustibles will burn in the space simultaneously. Instead, burning will be most intense in, or perhaps limited to, the combustibles nearest to a ventilation source. For modest-sized compartments with low aspect ratios, the temperature history of the design fire can be determined by algebraic equations or computer models, such as those described in the SFPE *Handbook of Fire Protection Engineering* (SFPE, 2002).

4.2.1.3. Exterior Fires

A design guide is available for determining the exposure resulting from an exterior fire (AISI, 1979).

4.2.1.4. Fire Duration

Caution should be exercised when determining the fire duration for spaces with high aspect ratios, for example, 5:1 or greater, or for large spaces, for example, those with a floor area in excess of 5,000 ft^2 (465 m^2). The principal difficulty lies in obtaining a realistic estimate for the mass loss rate, given that all combustibles within the space may not be burning simultaneously. Failure to recognize uneven burning will result in an overestimation of the mass burning rate and an underestimation of the fire duration by a significant margin. Note: some computation methods may implicitly determine the duration of the fire, in which case the calculation of mass loss rate is unnecessary.

Where a parametric curve is used to define a post-flashover fire, the duration is determined by means of the fuel versus ventilation provisions, not explicitly by loss of mass. This clause should not limit the use of temperature-time relationships to those where duration is calculated, as stated above, as these tend to be localized fires and external fire.

4.2.1.5. Active Fire Protection Systems

Due consideration should be given to the reliability and effectiveness of active fire protection systems when describing the design-basis fire. When an automatic sprinkler system is installed, the total fuel load may be reduced by up to 60 percent (Eurocode 1, 1991). The maximum reduction in the fuel load should be considered only when the automatic sprinkler system is considered to be of the highest reliability, for example, reliable and adequate water supply, supervision of control valves, regular schedule for maintenance of the automatic sprinkler system developed in accordance with NFPA (2002), or alterations of the automatic sprinkler system are considered any time alterations for the space are considered.

For spaces with automatic smoke and heat vents, computer models are available to determine the smoke temperature (SFPE, 2002). Reduction in the temperature profile as a result of smoke and heat vents should only be considered for reliable installations of smoke and heat vents. As such, a regular maintenance schedule for the vents needs to be established in accordance with NFPA (2002a).

4.2.2. Temperatures in Structural Systems under Fire Conditions

The heat transfer analysis may range from one-dimensional analyses where the steel is assumed to be at uniform temperature to three-dimensional analyses. The uniform temperature assumption is appropriate in a “lumped heat capacity analysis” where a steel column, beam or truss element is uniformly heated along the entire length and around the entire perimeter of the exposed section and the protection system is uniform along the entire length and around the entire perimeter of the section. In cases with nonuniform heating or where different protection methods are used on different sides of the column, a one-dimensional analysis should be conducted for steel column assemblies. Two-dimensional analyses are appropriate for beams, bar joists or truss elements supporting floor or roof slabs.

Heat transfer analyses should consider changes in material properties with increasing temperature for all materials included in the assembly. This may be done in the lumped heat capacity analysis using an effective property value, determined at a temperature near the estimated mid-point of the temperature range expected to be experienced by that component over the duration of the exposure. In the one- and two-dimensional analyses, the variation in properties with temperature should be explicitly included.

The boundary conditions for the heat transfer analysis shall consider radiation heat transfer in all cases and convection heat transfer if the exposed element is submerged in the smoke or is being subjected to flame impingement. The presence of fire resistive materials in the form of insulation, heat screens or other protective measures shall be taken into account, if appropriate.

Lumped Heat Capacity Analysis. This first-order analysis to predict the temperature rise of steel structural members can be conducted using algebraic equations iteratively. This approach assumes that the steel member has a uniform temperature, applicable to cases where the steel member is unprotected or uniformly protected (on all sides), and is exposed to fire around the entire perimeter of the assembly containing the steel member. Caution should be used when applying this method to steel beams supporting floor and roof slabs, as the approach will overestimate the temperature rise in the beam. In addition, where this analysis is used as input for the structural analysis of a fire-exposed, steel beam supporting a floor and roof slab, the thermally induced moments will not be simulated as a result of the uniform temperature assumption.

Unprotected steel members. The temperature rise in an unprotected steel section in a short time period shall be determined by

$$\Delta T_s = \frac{a}{c_s\left(\dfrac{W}{D}\right)}(T_F - T_s)\,\Delta t \qquad \text{(C-A-4-2-1)}$$

The heat transfer coefficient, a, is determined from

$$a = a_c + a_r \qquad \text{(C-A-4-2-2)}$$

where

a_c = convective heat transfer coefficient

a_r = radiative heat transfer coefficient, given as

$$a_r = \frac{5.67\times10^{-8}\varepsilon_F}{T_F - T_S}\left(T_F^4 - T_s^4\right)$$

For the standard exposure, the convective heat transfer coefficient, a_c, can be approximated as 25 W/m^2-°C. The parameter, ε_F, accounts for the emissivity

TABLE C-A-4-2.1
Guidelines for Estimating ε_F

Type of Assembly	ε_F
Column, exposed on all sides	0.7
Floor beam: Imbedded in concrete floor slab, with only bottom flange of beam exposed to fire	0.5
Floor beam, with concrete slab resting on top flange of beam	
Flange width : beam depth ratio $\geq$ 0.5	0.5
Flange width : beam depth ratio $<$ 0.5	0.7
Box girder and lattice girder	0.7

of the fire and the view factor. Estimates for ε_F, are suggested in Table C-A-4-2.1.

For accuracy reasons, a maximum limit for the time step, Δt, is suggested as 5 sec.

The fire temperature needs to be determined based on the results of the design fire analysis. As alternatives, the standard time-temperature curves indicated in ASTM E119 (ASTM, 2000) for building fires or ASTM E1529 (ASTM, 2000a) for petrochemical fires may be selected.

Protected Steel Members. This method is most applicable for steel members with contour protection schemes, in other words, where the insulating or (protection) material follows the shape of the section. Application of this method for box protection methods will generally result in the temperature rise being overestimated. The approach assumes that the outside insulation temperature is approximately equal to the fire temperature. Alternatively, a more complex analysis may be conducted which determines the exterior insulation temperature from a heat transfer analysis between the assembly and the exposing fire environment.

If the thermal capacity of the insulation is much less than that for the steel, such that the following inequality is satisfied:

$$c_s\, W/D > 2d_p\, \rho_p\, c_p \tag{C-A-4-2-3}$$

Then, Equation C-A-4-2-4 can be applied to determine the temperature rise in the steel:

$$\Delta T_s = \frac{k_p}{c_s d_p \dfrac{W}{D}}\,(T_F - T_s)\,\Delta t \tag{C-A-4-2-4}$$

If the thermal capacity of the insulation needs to be considered (such that the inequality in Equation C-A-4-2-3 is not satisfied), then Equation C-A-4-2-5 should be applied:

$$\Delta T_s = \frac{k_p}{d_p}\left[\frac{T_F - T_s}{c_s \dfrac{W}{D} + \dfrac{c_p \rho_p d_p}{2}}\right]\Delta t \tag{C-A-4-2-5}$$

The maximum limit for the time step, Δt, should be 5 sec.

Ideally, material properties should be considered as a function of temperature. Alternatively, material properties may be evaluated at a mid-range temperature expected for that component. For protected steel members, the material properties may be evaluated at 300 °C, and for protection materials, a temperature of 500 °C may be considered.

External Steelwork. Temperature rise can be determined by applying the following equation:

$$\Delta T_s = \frac{q''}{c_s\left(\dfrac{W}{D}\right)}\Delta t \tag{C-A-4-2-6}$$

where q'' is the net heat flux incident on the steel member

Advanced Calculation Methods. The thermal response of steel members may be assessed by application of a computer model. A computer model for analyzing the thermal response of the steel members should consider the following:

- Exposure conditions established based on the definition of a design fire. The exposure conditions need to be stipulated either in terms of a time-temperature history, along with radiation and convection heat transfer parameters associated with the exposure, or as an incident heat flux. The incident heat flux is dependent on the design fire scenario and the location of the structural assembly. The heat flux emitted by the fire or smoke can be determined from a fire hazard analysis. Exposure conditions are established based on the definition of a design fire. The exposure conditions are stipulated either in terms of a time-temperature history, along with radiation and convection heat transfer parameters associated with the exposure, or as an incident heat flux.
- Temperature-dependent material properties.
- Temperature variation within the steel member and any protection components, especially where the exposure varies from side to side.

Nomenclature:

A_m	surface area of a member per unit length, ft (m)
A_p	area of the inner surface of the fire protection material per unit length of the member, ft (m)
A_c	cross-sectional area, in.^{2} (m^2)
D	heat perimeter, in. (m)
T	temperature, °F (°C)
V	volume of a member per unit length, in.^{2} (m^2)
W	weight (mass) per unit length, lb/ft (kg/m)
a	heat transfer coefficient, Btu/ft^2·sec·°F (W/m^2·°C)
c	specific heat, Btu/lb·°F (J/kg·°C)
d	thickness, in. (m)

$h_{net,d}$	design value of the net heat flux per unit area, Btu/sec·ft^2 (W/m^2)
k	thermal conductivity, Btu/ft·sec·°F (W/m·°C)
l	length, ft (m)
t	time in fire exposure, seconds
Δt	time interval, seconds
ρ	density, lb/ft^3 (kg/m^3)

Subscripts:

a	steel
c	convection
m	member
p	fire protection material
r	radiation
s	steel
t	dependent on time
T	dependent on temperature

4.2.3. Material Strengths at Elevated Temperatures

The properties for steel and concrete at elevated temperatures are adopted from the ECCS *Model Code on Fire Engineering* (ECCS, 2001), Section III.2, "Material Properties." These generic properties are consistent with those in Eurocodes 3 (Eurocode 3, 2002) and 4 (Eurocode 4, 2003), and reflect the consensus of the international fire engineering and research community. The background information for the mechanical properties of structural steel at elevated temperatures can be found in Cooke (1988) and Kirby and Preston (1988).

4.2.4. Structural Design Requirements

The resistance of the structural system in the design basis fire may be determined by:

(a) Structural analysis of individual elements where the effects of restraint to thermal expansion and bowing may be ignored but the reduction in strength and stiffness with increasing temperature is incorporated.
(b) Structural analysis of assemblies/subframes where the effects of restrained thermal expansion and thermal bowing are considered by incorporating geometric and material nonlinearities.
(c) Global structural analysis where restrained thermal expansion, thermal bowing, material degradation and geometric nonlinearity are considered.

4.2.4.1. General Structural Integrity

The requirement for general structural integrity is consistent with that appearing in Section 1.4 of ASCE (2002). Structural integrity is the ability of the structural system to absorb and contain local damage or failure without developing into a progressive collapse that involves the entire structure or a disproportionately large part of it.

The Commentary C1.4 to Section 1.4 of ASCE (2002) contains guidelines for the provision of general structural integrity. Compartmentation (subdivision of buildings/stories in a building) is an effective means of achieving resistance to progressive collapse as well as preventing fire spread, as a cellular arrangement of structural components that are well tied together provides stability and integrity to the structural system as well as insulation.

4.2.4.2. Strength Requirements and Deformation Limits

As structural elements are heated, their expansion is restrained by adjacent element and connections. Material properties degrade with increasing temperature. Load transfer can occur from hotter elements to adjacent cooler elements. Excessive deformation may be of benefit in a fire as it allows release of thermally induced stresses. Deformation is acceptable once horizontal and vertical separation as well as the overall load bearing capacity of the structural system is maintained.

4.2.4.3. Methods of Analysis

4.2.4.3a. Advanced Methods of Analysis

Advanced methods are required when the overall structural system response to fire, the interaction between structural members and separating elements in fire, or the residual strength of the structural system following a fire must be considered.

4.2.4.3b. Simple Methods of Analysis

Simple methods may suffice when a structural member or component can be assumed to be subjected to uniform heat flux on all sides and the assumption of a uniform temperature is reasonable as, for example, in a free-standing column.

4.2.4.4. Design Strength

The design strength for structural steel members and connections is calculated as ϕR_n, in which $R_n =$ nominal strength, in which the deterioration in strength at elevated temperature is taken into account, and ϕ is the resistance factor. The nominal strength is computed as in Chapters C, D, E, F, G, H, I, J and K of the Specification, using material strength and stiffnesses at elevated temperatures defined in Tables A-4.2.1 and A-4.2.2. While ECCS (2001) and Eurocode 1 (1991) specify partial material factors as equal to 1.0 for "accidental" limit states, the uncertainties in strength at elevated temperatures are substantial and in some cases are unknown. Accordingly, the resistance factors herein are the same as those at ordinary conditions.

4.3. DESIGN BY QUALIFICATION TESTING

Qualification testing is an acceptable alternative to design by analysis for providing fire resistance. It is anticipated that the basis will be ASCE (1998), ASTM (2000) and similar documents.

An unrestrained condition is one in which expansion at the support of a load carrying element is not resisted by forces external to the element and the supported ends are free to expand and rotate. A steel member bearing on a wall in a single span or at the end span of multiple spans should be considered unrestrained when the wall has not been designed and detailed to resist thermal thrust.

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APPENDIX 5

EVALUATION OF EXISTING STRUCTURES

5.1. GENERAL PROVISIONS

The load combinations referred to in this chapter pertain to gravity loading because it is the most prevalent condition encountered. If other loading conditions are a consideration, such as lateral loads, the appropriate load combination from ASCE (2002) or from the applicable building code should be used. The engineer of record for a project is generally established by the owner.

5.2. MATERIAL PROPERTIES

1. Determination of Required Tests

The extent of tests required depends on the nature of the project, the criticality of the structural system or member evaluated, and the availability of records pertinent to the project. Thus, the engineer of record has the responsibility to determine the specific tests required and the locations from which specimens are to be obtained.

2. Tensile Properties

Samples required for tensile tests should be removed from regions of reduced stress, such as at flange tips at beam ends and external plate edges, to minimize the effects of the reduced area. The number of tests required will depend on whether they are conducted to merely confirm the strength of a known material or to establish the strength of some other steel.

It should be recognized that the yield stress determined by standard ASTM methods and reported by mills and testing laboratories is somewhat greater than the static yield stress because of dynamic effects of testing. Also, the test specimen location may have an effect. These effects have already been accounted for in the nominal strength equations in the Specification. However, when strength evaluation is done by load testing, this effect should be accounted for in test planning because yielding will tend to occur earlier than otherwise anticipated. The static yield stress, F_{ys}, can be estimated from that determined by routine application of ASTM methods, F_y, by the following equation (Galambos, 1978; Galambos, 1998):

$$F_{ys} = R(F_y - 4) \quad \text{(C-A-5-2-1)}$$

$$[\text{S.I.}: F_{ys} = R(F_y - 27)] \quad \text{(C-A-5-2-1M)}$$

where

F_{ys} = static yield stress, ksi (MPa)
F_y = reported yield stress, ksi (MPa)
R = 0.95 for tests taken from web specimens
= 1.00 for tests taken from flange specimens

The R factor in Equation C-A-5-2-1 accounts for the effect of the coupon location on the reported yield stress. Prior to 1997, certified mill test reports for structural shapes were based on specimens removed from the web, in accordance with ASTM A6/A6M (ASTM, 2003). Subsequently the specified coupon location was changed to the flange. During 1997–1998, there was a transition from web specimens to flange specimens as the new provisions of ASTM A6/A6M (ASTM, 2003) were adopted.

4. Base Metal Notch Toughness

The engineer of record shall specify the location of samples. Samples shall be cored, flame cut or saw cut. The engineer of record will determine if remedial actions are required, such as the possible use of bolted splice plates.

5. Weld Metal

Because connections typically are more reliable than structural members, strength testing of weld metal is not usually necessary. However, field investigations have sometimes indicated that complete-joint-penetration groove welds, such as at beam-to-column connections, were not made in accordance with AWS D1.1 (AWS, 2004). The specified provisions in AWS D1.1, Section 5.2.4 provide a means for judging the quality of such a weld. Where feasible, any samples removed should be obtained from compression splices rather than tension splices, because the effects of repairs to restore the sampled area are less critical.

6. Bolts and Rivets

Because connections typically are more reliable than structural members, removal and strength testing of fasteners is not usually necessary. However, strength testing of bolts is required where they can not be properly identified otherwise. Because removal and testing of rivets is difficult, assuming the lowest rivet strength grade simplifies the investigation.

5.3. EVALUATION BY STRUCTURAL ANALYSIS

2. Strength Evaluation

Resistance and safety factors reflect variations in determining strength of members and connections, such as uncertainty in theory and variations in material properties and dimensions. If an investigation of an existing structure indicates that there are variations in material properties or dimensions significantly greater than those anticipated in new construction, the engineer of record should consider the use of more conservative values.

5.4. EVALUATION BY LOAD TESTS

1. Determination of Live Load Rating by Testing

Generally, structures that can be designed according to the provisions of this Specification need no confirmation of calculated results by test. However, special situations may arise when it is desirable to confirm by tests the results of calculations. Minimal test procedures are provided to determine the live load rating of a structure. However, in no case is the live load rating determined by test to exceed that which can be calculated using the provisions of this Specification. This is not intended to preclude testing to evaluate special conditions or configurations that are not adequately covered by this Specification.

It is essential that the engineer of record take all necessary precautions to ascertain that the structure does not fail catastrophically during testing. A careful assessment of structural conditions before testing is a fundamental requirement. This includes accurate measurement and characterization of the size and strength of members, connections and details. All safety regulations of OSHA and other pertinent bodies must be strictly adhered to. Shoring and scaffolding should be used as required in the proximity of the test area to mitigate against unexpected circumstances. Deformations must be carefully monitored and structural conditions must be continually evaluated. In some cases it may be desirable to monitor strains as well.

The engineer of record must use judgment to determine when deflections are becoming excessive and terminate the tests at a safe level even if the desired loading has not been achieved. Incremental loading is specified so that deformations can be accurately monitored and the performance of the structure carefully observed. Load increments should be small enough initially so that the onset of significant yielding can be determined. The increment can be reduced as the level of inelastic behavior increases, and the behavior at this level carefully evaluated to determine when to safely terminate the test. Periodic unloading after the onset of inelastic behavior will help the engineer of record determine when to terminate the test to avoid excessive permanent deformation or catastrophic failure.

It must be recognized that the margin of safety at the maximum load level used in the test may be very small, depending on such factors as the original design, the purpose of the tests, and the condition of the structure. Thus, it is imperative that all appropriate safety measures be adopted. It is recommended that the maximum live load used for load tests be selected conservatively. It should be noted that experience in testing more than one bay of a structure is limited.

The provision limiting increases in deformations for a period of one hour is given so as to have positive means that the structure is stable at the loads evaluated.

2. Serviceability Evaluation

In certain cases serviceability performance must be determined by load testing. It should be recognized that complete recovery (in other words, return to initial deflected shape) after removal of maximum load is unlikely because of phenomena such as local yielding, slip at the slab interface in composite construction, creep in concrete slabs, localized crushing or deformation at shear connections in slabs, slip in bolted connections, and effects of continuity. Because most structures exhibit some slack when load is first applied, it is appropriate to project the load-deformation curve back to zero load to determine the slack and exclude it from the recorded deformations. Where desirable, the applied load sequence can be repeated to demonstrate that the structure is essentially elastic under service loads and that the permanent set is not detrimental.

5.5. EVALUATION REPORT

Extensive evaluation and load testing of existing structures is often performed when appropriate documentation no longer exists or when there is considerable disagreement about the condition of a structure. The resulting evaluation is only effective if well documented, particularly when load testing is involved. Furthermore, as time passes, various interpretations of the results can arise unless all parameters of the structural performance, including material properties, strength, and stiffness, are well documented.

APPENDIX 6

STABILITY BRACING FOR COLUMNS AND BEAMS

6.1. GENERAL PROVISIONS

The design requirements of Appendix 6 consider two general types of bracing systems, relative and nodal, as shown in Figure C-A-6.1.

A relative column brace system (such as diagonal bracing or shear walls) is attached to two locations along the length of the column that defines the unbraced length. The relative brace system shown consists of the diagonal and the strut that controls the movement at one end of the unbraced length, A, with respect to the other end of the unbraced length, B. The diagonal and the strut both contribute to the strength and stiffness of the relative brace system. However, when the strut is a floor beam, its stiffness is large compared to the diagonal so the diagonal controls the strength and stiffness of the relative brace.

A nodal brace controls the movement only at the particular brace point, without direct interaction with adjacent braced points. Therefore to define an unbraced length, there must be additional adjacent brace points as shown in Figure C-A-6.1. The two nodal column braces at C and D that are attached to the rigid abutment define the unbraced length for which $K = 1.0$ can be used. For beams a cross frame between two adjacent beams at midspan is a nodal brace because it prevents twist of the beams only at the particular cross frame location. The unbraced length is half the span length. The twist at the ends of the two beams is prevented by the beam-to-column connections at the end supports. Similarly, a nodal lateral brace attached at midspan to the top flange of the beams and a rigid support assumes that there is no lateral movement at the column locations.

The brace requirements are intended to enable a member to potentially reach a maximum load based on the unbraced length between the brace points and $K = 1.0$. This is not the same as the no-sway buckling load as illustrated in Figure C-A-6.2 for a braced cantilever. The critical stiffness is 1.0 P_e/L, corresponding to $K = 1.0$. A brace with five times this stiffness is necessary to reach 95 percent of the $K = 0.7$ limit. Theoretically, an infinitely stiff brace is required to reach the no-sway limit. Bracing required to reach specified rotation capacities or ductility limits is beyond the scope of these recommendations. Member inelasticity has no significant effect on the brace requirements (Yura, 1995).

Winter developed the concept of a dual requirement for bracing design: strength and stiffness (Winter, 1958; Winter, 1960). The brace force is a function of the initial column out-of-straightness, Δ_o, and the brace stiffness, β. For a relative brace system, the relationship between column load, brace stiffness and sway displacement is shown in Figure C-A-6.3. If $\beta = \beta_i$, the critical brace stiffness for a perfectly plumb member, then $P = P_e$ only if the sway deflection gets very large. Unfortunately, such large displacements produce large brace forces. For practical design, Δ must be kept small at the factored load level.

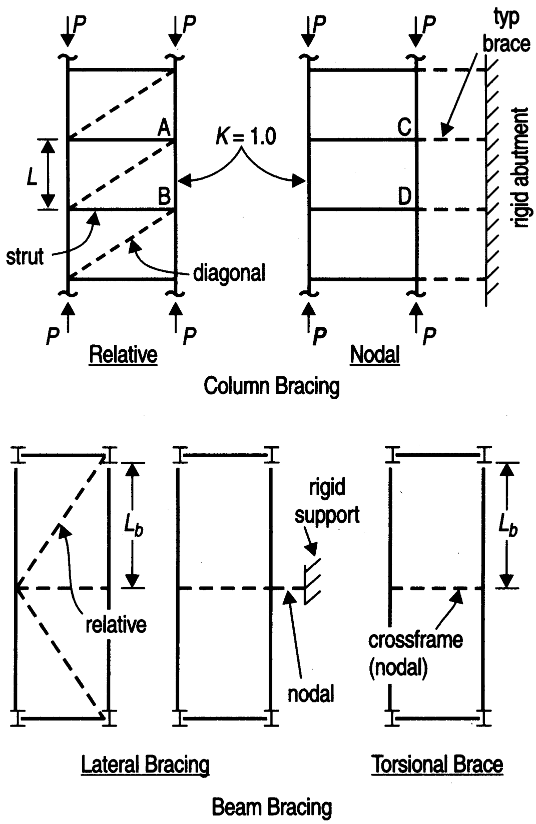


Fig. C-A-6.1. Types of bracing.

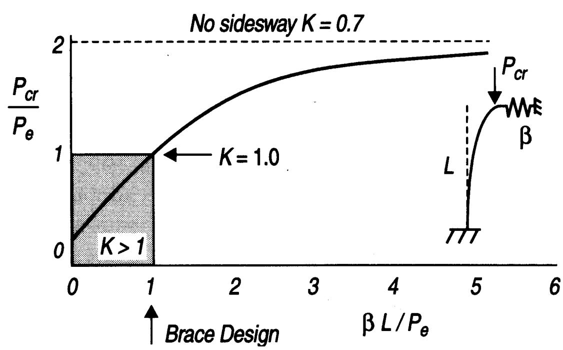


Fig. C-A-6.2. Braced cantilever.

The brace stiffness requirements, β_{br}, for frames, columns, and beams were chosen as twice the critical stiffness. All brace stiffness requirements use a $\phi = 0.75$. For the relative brace system shown in Figure C-A-6.3, $\beta_{br} = 2\beta_i$ gives $P_{br} = 0.4\% P_e$ for $\Delta_o = 0.002L$. If the brace stiffness provided, β_{act}, is different from the requirement, then the brace force or brace moment can be multiplied by the following factor:

$$\frac{1}{2 - \dfrac{\beta_{br}}{\beta_{act}}} \qquad \text{(C-A-6-1)}$$

No ϕ is specified in the brace strength requirements since ϕ is included in the component design strength provisions in other chapters of this Specification.

The initial displacement, Δ_o, for relative and nodal braces is defined with respect to the distance between adjacent braces, as shown in Figure C-A-6.4. The initial Δ_o is a displacement from the straight position at the brace points caused by sources other than brace elongations from gravity loads or compressive forces, such as displacements caused by wind or other lateral forces, erection tolerances, column

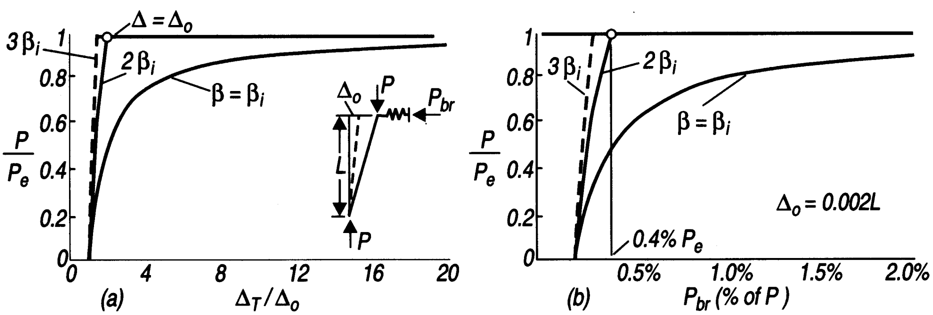


Fig. C-A-6.3. Effect of initial out-of-plumbness.

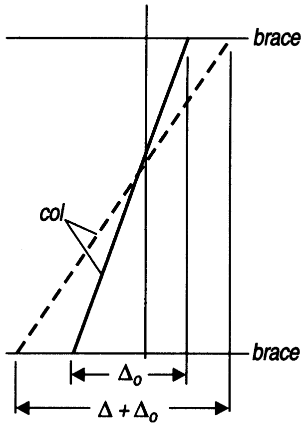


Fig. C-A-6.4. Definitions of initial displacements for relative and nodal braces.

shortening, etc. The brace force recommendations for frames, columns and beam lateral bracing are based on an assumed $\Delta_o = 0.002L$, where L is the distance between adjacent brace points. For torsional bracing of beams, an initial twist angle, θ_o, is assumed where $\theta_o = 0.002L/h_o$, and h_o is the distance between flange centroids. For other Δ_o and θ_o values, use direct proportion to modify the brace strength requirements, P_{br} and M_{br}. For cases where it is unlikely that all columns in a story are out-of-plumb in the same direction, Chen and Tong recommend an average $\Delta_o = 0.002L/\sqrt{n_o}$ where n_o columns, each with a random Δ_o, are to be stabilized by the brace system (Chen and Tong, 1994). This reduced Δ_o would be appropriate when combining the stability brace forces with wind and seismic forces.

Brace connections, if they are flexible or can slip, should be considered in the evaluation of the bracing stiffness as follows:

$$\frac{1}{\beta_{act}} = \frac{1}{\beta_{conn}} + \frac{1}{\beta_{brace}} \tag{C-A-6-2}$$

The brace system stiffness, β_{act}, is less than the smaller of the connection stiffness, β_{conn}, or the stiffness of the brace, β_{brace}. Slip in connections with standard holes need not be considered except when only a few bolts are used. When evaluating the bracing of rows of columns or beams, consideration must be given to the accumulation of the brace forces along the length of the brace that results in a different displacement at each beam or column location. In general, brace forces can be minimized by increasing the number of braced bays and using stiff braces.

6.2. COLUMNS

For nodal column bracing, the critical stiffness is a function of the number of intermediate braces (Winter, 1958; Winter, 1960). For one intermediate brace, $\beta_i = 2P/L_b$, and for many braces $\beta_i = 4P/L_b$. The relationship between the critical stiffness and the number of braces, n, can be approximated (Yura, 1995) as $\beta_i = N_i P/L_b$, where $N_i = 4 - 2/n$. The most severe case (many braces) was adopted for the brace stiffness requirement, $\beta_{br} = 2 \times 4P/L_b$. The brace stiffness, Equation A-6-4, can be reduced by the ratio, $N_i/4$, to account for the actual number of braces.

The unbraced length, L_b, in Equation A-6-4 is assumed to be equal to the length L_q that enables the column to reach P_u. When the actual bracing spacing is less than L_q, the calculated required stiffness may become quite conservative since the stiffness equations are inversely proportional to L_b. In such cases, L_q can be substituted for L_b. (This substitution is also applicable for the beam nodal bracing formulations given in Equations A-6-8 and A-6-9.) For example, a W12×53 (W310×79) with P_u = 400 kips (1 780 kN) can have a maximum unbraced length of 14 ft (4.3 m) for A36 (A36M) steel. If the actual bracing spacing is 8 ft (2.4 m), then 14 ft (4.3 m) may be used in Equation A-6-4 to determine the required stiffness. The use of L_q in Equation A-6-4 provides reasonable estimates of the brace stiffness requirements; however, the solution can still result in conservative estimates of the stiffness requirements. Improved accuracy can be obtained by treating the system as a continuous bracing system as discussed in Galambos (1998) and Lutz and Fisher (1985).

With regards to the brace strength requirements, Winter's rigid model only accounts for force effects from lateral displacements and would derive a brace force of 0.8 percent P_u, which accounts only for lateral displacement force effects. To account for the additional force due to member curvature, this theoretical force has been increased to $1.0\% P_u$.

6.3. BEAMS

Beam bracing must prevent twist of the section, not lateral displacement. Both lateral bracing (for example, joists attached to the compression flange of a simply supported beam) and torsional bracing (for example, a cross frame or diaphragm between adjacent girders) can effectively control twist. Lateral bracing systems that are attached near the beam centroid are ineffective. For beams with double curvature, the inflection point can not be considered a brace point because twist occurs at that point (Galambos, 1998). A lateral brace on one flange near the inflection point also is ineffective. In double curvature cases the lateral brace near the inflection point must be attached to both flanges to prevent twist, or torsional bracing must be used. The beam brace requirements are based on the recommendations in Yura (1993).

1. Lateral Bracing

For lateral bracing, the following stiffness requirement was derived following Winter's approach:

$$\beta_{br} = 2N_i(C_b P_f)\, C_t C_d / \phi L_b \tag{C-A-6-3}$$

where

N_i = 1.0 for relative bracing
= $(4-2/n)$ for discrete bracing
n = number of intermediate braces
P_f = beam compressive flange force
= $\pi^2 EI_{yc}/L_b{}^2$
I_{yc} = out-of-plane moment of inertia of the compression flange
C_b = moment modifier from Chapter F
C_t = accounts for top flange loading (use $C_t = 1.0$ for centroidal loading)
= $1 + (1.2/n)$
C_d = double curvature factor (compression in both flanges)
= $1 + (M_S/M_L)^2$
M_S = smallest moment causing compression in each flange
M_L = largest moment causing compression in each flange

The C_d factor varies between 1.0 and 2.0 and is applied only to the brace closest to the inflection point. The term $(2N_i C_t)$ can be conservatively approximated as

10 for any number of nodal braces and 4 for relative bracing and $(C_b P_f)$ can be approximated by M_u/h which simplifies Equation C-A-6-3 to the stiffness requirements given by Equations A-6-6 and A-6-8. Equation C-A-6-3 can be used in lieu of Equations A-6-6 and A-6-8.

The brace strength requirement for relative bracing is

$$P_{br} = 0.004 M_u C_t C_d / h_o \qquad \text{(C-A-6-4a)}$$

and for nodal bracing

$$P_{br} = 0.01 M_u C_t C_d / h_o \qquad \text{(C-A-6-4b)}$$

They are based on an assumed initial lateral displacement of the compression flange of $0.002L_b$. The brace strength requirements of Equations A-6-5 and A-6-7 are derived from Equations C-A-6-4a and C-A-6-4b assuming top flange loading ($C_t = 2$). Equations C-A-6-4a and C-A-6-4b can be used in lieu of Equations A-6-5 and A-6-7, respectively.

2. Torsional Bracing

Torsional bracing can either be attached continuously along the length of the beam (for example, metal deck or slabs) or be located at discrete points along the length of the member (for example, cross frames). With respect to the girder response, torsional bracing attached to the tension flange is just as effective as a brace attached at mid-depth or to the compression flange. Although the girder response is generally not sensitive to the brace location, the position of the brace on the cross section does have an effect on the stiffness of the brace itself. For example, a torsional brace attached on the bottom flange will often bend in single curvature (for example, with a flexural stiffness of $2EI/L$ based on the brace properties), while a brace attached on the top flange will often bend in reverse curvature (for example, with a flexural stiffness of $6EI/L$ based on the brace properties). Partially restrained connections can be used if their stiffness is considered in evaluating the torsional brace stiffness.

The torsional brace requirements are based on the buckling strength of a beam with a continuous torsional brace along its length presented in Taylor and Ojalvo (1966) and modified for cross-section distortion in Yura (1993).

$$M_u \le M_{cr} = \sqrt{(C_{bu} M_o)^2 + \frac{C_b^2 E I_y \overline{\beta}_T}{2C_{tt}}} \qquad \text{(C-A-6-5)}$$

The term $(C_{bu} M_o)$ is the buckling strength of the beam without torsional bracing. $C_{tt} = 1.2$ when there is top flange loading and $C_{tt} = 1.0$ for centroidal loading. $\overline{\beta}_T = n\beta_T/L$ is the continuous torsional brace stiffness per unit length or its equivalent when n nodal braces, each with a stiffness β_T, are used along the span L and the 2 accounts for initial out-of-straightness. Neglecting the unbraced beam buckling term gives a conservative estimate of the torsional brace stiffness requirement (Equation A-6-11).

The strength requirements for beam torsional bracing were developed based upon an assumed initial twist imperfection of $\theta_o = 0.002L_b/h_o$, where h_o is equal to the depth of the beam. Providing at least twice the ideal stiffness results in a brace force, $M_{br} = \beta_T\theta_o$. Using the LRFD formulation of Equation A-6-11 (without ϕ), the strength requirement for the torsional bracing is

$$M_{br} = \beta_T\theta_o = \frac{2.4LM_u^2}{nEI_yC_b^2}\,\frac{L_b}{500h_o} \qquad \text{(C-A-6-6)}$$

To obtain Equation A-6-9, the equation was simplified as follows:

$$M_{br} = \frac{2.4LM_u^2}{nEI_yC_b^2}\,\frac{L_b}{500h_o}\,\frac{\pi^2L_b^2}{\pi^2L_b^2} = \frac{2.4\pi^2M_uL}{500nL_bC_b^2}\,\frac{M_u}{h_o}\,\frac{L_b^2}{C_b\pi^2EI_y} \qquad \text{(C-A-6-7)}$$

The term M_u/h_o can be approximated as the flange force, P_f, and the term $L_b^2/C_b\pi^2EI_y$ can be represented as the reciprocal of twice the buckling strength of the flange ($^1/_2P_f$). Substituting for these terms and evaluating the constants results in

$$M_{br} = \frac{0.024M_uL}{nC_bL_b} \qquad \text{(C-A-6-8)}$$

which is the expression given in Equation A-6-9.

Equations A-6-9 and A-6-12 give the strength and stiffness requirements for doubly symmetric beams. For singly symmetric sections these equations will generally be conservative. Better estimates of the strength requirements for torsional bracing of singly symmetric sections can be obtained with Equation C-A-6-6 by replacing I_y with I_{eff} as given in the following expression:

$$I_{eff} = I_{yc} + \frac{t}{c}I_{yt} \qquad \text{(C-A-6-9)}$$

where t is the distance from the neutral axis to the extreme tensile fibers, c is the distance from the neutral axis to the extreme compressive fibers, and I_{yc} and I_{yt} are the respective moments of inertia of compression and tension flanges about an axis through the web. Good estimates of the stiffness requirements of torsional braces for singly symmetric I-shaped beams may be obtained using Equation A-6-11 and replacing I_y with I_{eff} given in Equation C-A-6-9.

The β_{sec} term in Equations A-6-10, A-6-12 and A-6-13 accounts for cross-section distortion. A web stiffener at the brace point reduces cross-sectional distortion and improves the effectiveness of a torsional brace. When a cross frame is attached near both flanges or a diaphragm is approximately the same depth as the girder, then web distortion will be insignificant so β_{sec} equals infinity. The required bracing stiffness, β_{Tb}, given by Equation A-6-10 was obtained by solving the following expression that represents the brace system stiffness including distortion effects:

$$\frac{1}{\beta_T} = \frac{1}{\beta_{Tb}} + \frac{1}{\beta_{sec}} \qquad \text{(C-A-6-10)}$$

Parallel chord trusses with both chords extended to the end of the span and attached to supports can be treated like beams. In Equations A-6-5 through A-6-9, M_u may be taken as the maximum compressive chord force times the depth of the truss to determine the brace strength and stiffness requirements. Cross-section distortion effects, β_{sec}, need not be considered when full-depth cross frames are used for bracing. When either chord does not extend to the end of the span, consideration should be given to control twist near the ends of the span by the use of cross frames or ties.

APPENDIX 7

DIRECT ANALYSIS METHOD

Appendix 7, the direct analysis method, addresses a new method for the stability analysis and design of structural steel systems comprised of moment frames, braced frames, shear walls or combinations thereof (AISC-SSRC, 2003a). While the precise formulation of the method is unique to the AISC Specification, some of its features have similarities to other major design specifications around the world including the Eurocodes, the Australian Standard, the Canadian Standard and ACI 318.

The direct analysis method has been developed with the goal of more accurately determining the load effects in the structure in the analysis stage and eliminating the need for calculating the effective buckling length (K factor) for columns in the first term of the beam-column interaction equations. This method is, therefore, a major step forward in the design of steel moment frames from past editions of the Specification. In addition, the method can be used for the design of braced frames and combined frame systems. Thus, this one method can be used for the design of all types of steel framed structures used in practice. The method can be expanded in the future beyond its use as a second-order elastic analysis tool as presented here. For example, it can be applied with inelastic or plastic analysis. Also, it can be used in the analysis of composite structures, although this application is not explicitly addressed in this Specification.

Chapter C requires that the direct analysis method, as described herein, be used wherever the value of the *sidesway* amplification ratio $\Delta_{2nd\ order}/\Delta_{1st\ order}$ (or B_2 from Equation C2-3), determined from a first-order analysis of the structure, exceeds 1.5. The method may also be used in lieu of the methods described in Chapter C for the analysis and design of any lateral load resisting frame in a steel building.

7.1. GENERAL REQUIREMENTS

There are potentially many parameters and behavioral effects that influence the stability of steel-framed structures (Birnstiel and Iffland, 1980; McGuire, 1992; White and Chen, 1993; ASCE Task Committee on Effective Length, 1997; Deierlein and White, 1998). Three of the most important aspects of stability behavior include geometric nonlinearities, spread-of-plasticity, and member limit states. These aspects ultimately govern frame deformations under applied loads and the resulting load effects in the structure.

Geometric Nonlinearities and Imperfections. Modern stability design provisions are based on the premise that the member forces are calculated by second-order elastic analysis, where equilibrium is satisfied on the deformed geometry of the structure. The amplification of first-order analysis forces by the traditional B_1 and B_2 factors in Chapter C is one method of conducting an approximate second-order elastic analysis. Where stability effects are significant, consideration must be given to initial geometric imperfections in the structure due to fabrication and erection tolerances. In the development and calibration of the direct analysis method, initial geometric imperfections are conservatively assumed to be equal to the maximum fabrication and erection tolerances permitted by the AISC *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2005). For columns and frames, this implies a member out-of-straightness equal to $L/1000$, where L is the member length between brace or framing points, and a frame out-of-plumbness equal to $H/500$, where H is the story height. The out-of-plumbness also may be limited by the absolute bounds specified in the *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2005).

Spread of Plasticity. The direct analysis method is also calibrated against inelastic distributed-plasticity analyses that account for the spread of plasticity through the member cross-section and along the member length. The nominal thermal *residual stresses* in W-shape members are assumed to have a maximum value of $0.3F_y$ in compression at the flange tips and to be distributed according to the so-called Lehigh pattern—a linear variation across the flanges and uniform tension in the web (Deierlein and White, 1998).

Member Limit States. Member strength may be controlled by one or more of the following limit states: cross-section yielding, local buckling, flexural buckling, and lateral-torsional or *flexural-torsional buckling*. For beam-columns in single axis flexure and compression, the analysis results from the direct analysis method may be used with the new interaction equations in Chapter H, which address in-plane flexural buckling and out-of-plane lateral torsional instability separately. The separate interaction equations reduce the conservatism in the 1999 *LRFD Specification* (AISC, 2000b) provisions, which combine the two limit state checks into one equation, by using the most severe combination of in-plane or out-of-plane limits for $P_u/\phi P_n$ and $M_u/\phi M_n$. A significant advantage of the direct analysis method is that the in-plane check with P_n in the interaction equation is determined using $K = 1.0$ (in other words, $KL = L$).

Second-Order Analysis. The stability design provisions of Chapter C are developed for use with second-order elastic analysis. It is important that all component and connection deformations that contribute to the lateral displacement of the structure be considered in the analysis. In practice, there are alternative approaches one can employ for conducting second-order analyses, some of which are more rigorous than others.

Rigorous second-order analyses are those that accurately model all significant second-order effects. Rigorous analyses include solution of the governing differential equation, either through stability functions or computer frame analysis programs that model these effects (McGuire, 1992; Deierlein and White, 1998).

Many (but not all) modern commercial computer programs are capable of rigorous analyses, although this should be verified by the user for each particular program. Methods that modify first-order analysis results through second-order amplifiers (for example, B_1 and B_2 factors) are in some cases accurate enough to constitute a rigorous analysis. The use of the B_1 and B_2 amplifiers is permitted, even when $B_2 > 1.5$, provided they are determined using the reduced stiffnesses defined in Equations A-7-2 and A-7-3.

Approximate second-order analyses are those that do not meet the requirements of rigorous analysis. A common type of approximate analysis is one that captures only P-Δ effects due to member end translations (for example, *interstory drift*) but fails to capture P-δ effects due to curvature of the member relative to its chord. Where P-δ effects are significant, errors arise in approximate methods that do not accurately account for the effect of P-δ moments on amplification of both local member moments and the global (Δ) displacements. These errors can occur both with second-order computer analysis programs and with the B_1 and B_2 amplifiers. (Maleck and White, 2003) suggest an equation equivalent to Equation A-7-1 to distinguish cases where P-δ effects can be safely ignored. Alternatively, the engineer should verify the accuracy of the second-order analysis by comparisons to known solutions for conditions similar to those in the structure. Examples of the errors one may encounter are discussed in LeMessurier (1977) and Deierlein and White (1998).

It is suggested that in most building structures, the second-order *sidesway* amplification (or the equivalent B_2), calculated with the reduced stiffness, should be kept no greater than $\Delta_{2nd\ order}/\Delta_{1st\ order} = 2.5$. At larger amplification levels, small changes in gravity loads or stiffnesses result in relatively large changes in *sidesway* deflections and internal second-order forces, due to large geometric nonlinearities. Also note that stiffness requirements for control of seismic drift are included in many building codes that prohibit amplification or B_2 levels from exceeding approximately 1.5 to 1.6 (typically calculated, for steel structures, without use of a reduced stiffness) (ICC, 2003).

Effective Length Method versus the Direct Analysis Method. The effective length method for assessing member axial compressive strength, as discussed in Chapter C of this Commentary, has been used in various forms in the AISC Specification since 1961. The provisions of the current Chapter C are essentially the same as those in the 1999 *LRFD Specification* (AISC, 2000b), with the exception that: (1) limits are placed on the magnitude of second-order effects (as quantified by the ($\Delta_{2nd\ order}/\Delta_{1st\ order}$) or B_2 limit of 1.5); and (2) a minimum lateral load of $0.002Y_i$ (where Y_i is the design gravity load acting on level i) is required to be placed at each level of the structure for all gravity load-only combinations. These limits and requirements are specified for the effective length method (which uses the nominal geometry and elastic stiffness) to limit errors caused by not explicitly accounting in the analysis for initial out-of-plumbness and member stiffness reduction due to spread of plasticity. The method is based on calculating effective column buckling lengths, KL, which have their basis in elastic (or inelastic) stability theory. In the effective length method, the effective buckling length KL, or alternatively the equivalent elastic column buckling load, $P_e = \pi^2 EI/(KL)^2$, is used to calculate an axial compressive strength, P_n, through an empirical *column curve* that accounts for geometric imperfections and distributed yielding (including *residual stress* effects). This column strength is then combined with the flexural strength, M_n, and second-order member forces, P_u and M_u, in the beam-column interaction equations.

Differences between the effective length method and the direct analysis method lie predominantly in the in-plane strength check. Figure C-A-7.1(a) shows a plot of the in-plane interaction equation for the effective length method, where the anchor point on the vertical axis, P_{nKL}, is determined using an effective buckling length. Also shown in this plot is the same interaction equation with the first term based on the yield load, P_y. For W-shape members, this in-plane beam-column interaction equation is a reasonable estimate of the internal force state associated with full cross-section plastification. The P versus M response of a typical member, obtained from second-order spread-of-plasticity analysis and labeled "actual response," indicates the maximum axial force, P_u, that the member can sustain prior to the onset of instability. The load-deflection response from a second-order elastic analysis using the nominal geometry and elastic stiffness, as conducted with the effective length method, is also shown. The "actual response" curve has larger moments than the above second-order elastic curve due to the combined effects of distributed yielding and geometric imperfections, which are not included in the second-order elastic analysis. In the effective length method, the intersection of the second-order elastic analysis curve with the P_{nKL} interaction curve determines the member strength. The plot in Figure C-A-7.1(a) shows that the effective length method is calibrated to give a resultant axial strength, P_u, consistent with the actual response. For slender columns, the calculation of the effective length KL (and P_{nKL}) is critical to achieving an accurate solution when using the effective length method.

While the effective length method is calibrated to accurately assess the resultant in-plane member strength, one consequence of the procedure is that it underestimates the actual internal moments under the factored loads (see Figure C-A-7.1(a)). This is inconsequential for the beam-column in-plane strength check (since P_{nKL} reduces the effective strength in the correct proportion); however, the reduced moment can affect the design of the beams and connections, which provide rotational restraint to the column. This is of greatest concern when the calculated moments are small and axial loads are large, such that P-Δ moments induced by column out-of-plumbness can be significant.

A major advantage of the direct analysis method is that it more accurately captures the internal forces in the structure, particularly for the cases where there are high gravity loads and low lateral loads. This advantage comes at the expense of applying notional lateral loads to the structure and reducing the frame stiffness as part of the analysis input.

7.2. NOTIONAL LOADS

Notional loads are lateral loads that are applied at each framing level and are specified in terms of the gravity loads applied at that level. The gravity loads used to determine the notional load must be equal to or greater than the gravity loads associated with the load combination being evaluated. Notional loads must be applied in the direction that adds to the destabilizing effects under the specified load combination.

The purpose of notional loads is to account for the destabilizing effects of geometric imperfections, nonideal conditions (such as incidental patterned gravity load effects, temperature gradients across the structure, foundation settlement, uneven column shortening, or any other effects that could induce sway that is not explicitly considered in the analysis), inelasticity in structural members, or

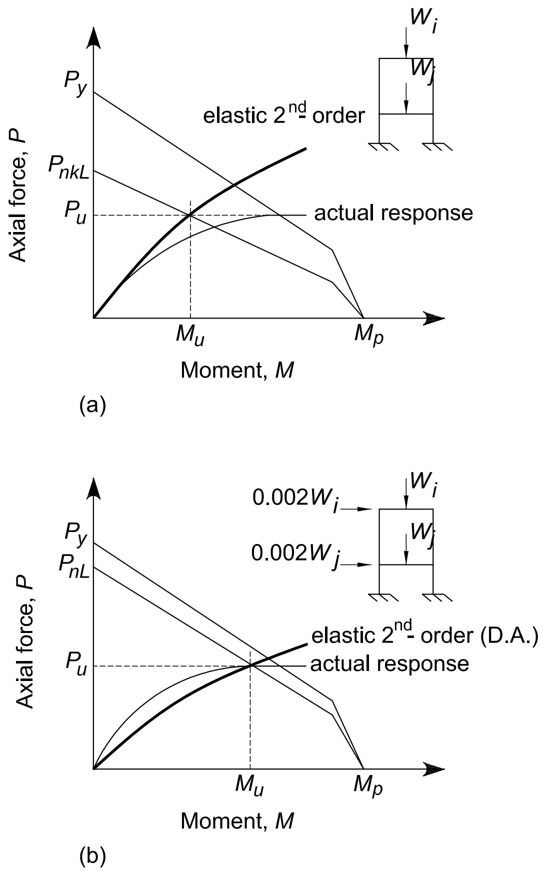


Fig. C-A-7.1. Comparison of in-plane beam-column interaction checks for (a) the effective length method and (b) the direct analysis method.

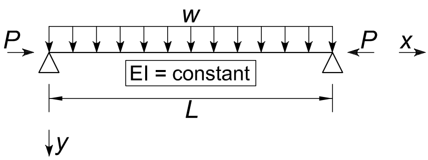
combinations thereof. While it accounts for any or all of these potential effects, the magnitude of the notional load $0.002Y_i$ can be thought of as representing an initial out-of-plumbness in each story of the structure of 1/500 times the story height. If a smaller value can be justified by the designer, it is permitted to adjust the magnitude of the notional load proportionately. Note that it is also permissible to model the structure in an assumed out-of-plumb state in lieu of applying the notional load.

7.3. DESIGN-ANALYSIS CONSTRAINTS

The direct analysis method begins with the basic requirement to calculate accurately the internal load effects using a rigorous second-order analysis. This stipulation is placed on the method to afford the luxury of using $K = 1.0$ in the first term of the beam-column interaction equation. In order to obtain accuracy in the calculation of second-order effects, certain constraints must be placed on the method as discussed below.

The first constraint (clause 1) requires that a rigorous second-order analysis be conducted that accounts for both P-Δ and P-δ effects. P-Δ effects are the effects of loads acting on the displaced location of joints or nodes in a structure. P-δ effects are the effect of loads acting on the deflected shape of a member between joints or nodes. Two benchmark problems have been established to determine whether an analysis method meets the requirements of a rigorous second-order analysis adequate for use in the direct analysis method. The problem descriptions and their rigorous differential equation solutions are shown in Figure C-A-7.2. Case 1 is a simply supported beam column subject to a uniform transverse load between supports. This problem contains only a P-δ effect since there is no translation of one end of the member relative to the other. The second problem is a flagpole column with a lateral load at its top. This problem contains both P-Δ and P-δ effects. Figure C-A-7.3 plots the results for the maximum moment and deflection as a function of the applied load P/P_{eL} using the rigorous solution. Note also that if the magnitude of the axial load on the member is less than or equal to $0.15P_{eL}$(where $P_{eL} = \pi^2 EI/L^2$), then it is permitted to ignore the P-δ effect on the lateral displacement Δ of the structure as the error in doing so is relatively small (Maleck and White, 2003). However, the P-δ effect on the internal moment in the member must be considered (see Figures C-A-7.2 and C-A-7.3). When using the benchmark problems to assess the correctness of a second-order analysis method or computer program, the computer model should utilize joints only at the ends of the member (unless joints are planned on being used along the member length in the actual structure to be modeled). Both moments and deflections should be checked at the location shown for various levels of axial load on the member (including loads that result in moment and deflection amplification, M_{max}/M_o and y_{max}/y_o, of more than 2.5) the results should agree within 3 percent. Other possible benchmark problems can be found in Chen and Lui (1987), which contains the rigorous solution for a simply-supported beam-column subject to compression and applied end moments and also a solution for a fixed-ended beam-column subject to compression and uniformly distributed loads. Typically, the calculation of accurate internal M_r values is more difficult in problems where member load and/or displacement boundary conditions are not symmetrical.

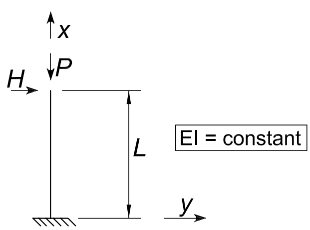
The second constraint (clause 2) requires the application of a notional load $N_i = 0.002Y_i$, where Y_i is the gravity load from the appropriate load combination acting on level i. The notional loads are required to account for the destabilizing effects of initial imperfections and other conditions that may induce sway not explicitly modeled in the structure. Note that the notional load coefficient 0.002 is based on an initial out-of-plumbness ratio from all effects equal to 1/500. Where a different value can be justified, the coefficient may be adjusted proportionately. When second-order effects are kept to a level so that the *sidesway* amplification $\Delta_{2nd\ order}/\Delta_{1st\ order}$ or $B_2 \leq 1.5$ (1.71 using the reduced elastic stiffness), then it is permitted to apply the notional loads only in the gravity load-only combinations and not in combination with other lateral loads. At this low range of *sidesway*



$$M_{MAX}\left(@x = \frac{L}{2}\right) = \frac{wL^2}{8}\left[\frac{2(\sec u - 1)}{u^2}\right] \quad \text{where } u = \sqrt{\frac{PL^2}{4EI}},\ M_o = \frac{wL^2}{8}$$

$$y_{MAX}\left(@x = \frac{L}{2}\right) = \frac{5wL^4}{384EI}\left[\frac{12(2\sec u - u^2 - 2)}{5u^4}\right] \quad \text{where } y_o = \frac{5wL^4}{384EI}$$

Case 1



$$M_{MAX}\,(@x = 0) = HL\left(\frac{\tan\alpha}{\alpha}\right) \quad \text{where } \alpha = \sqrt{\frac{PL^2}{EI}},\ M_o = HL$$

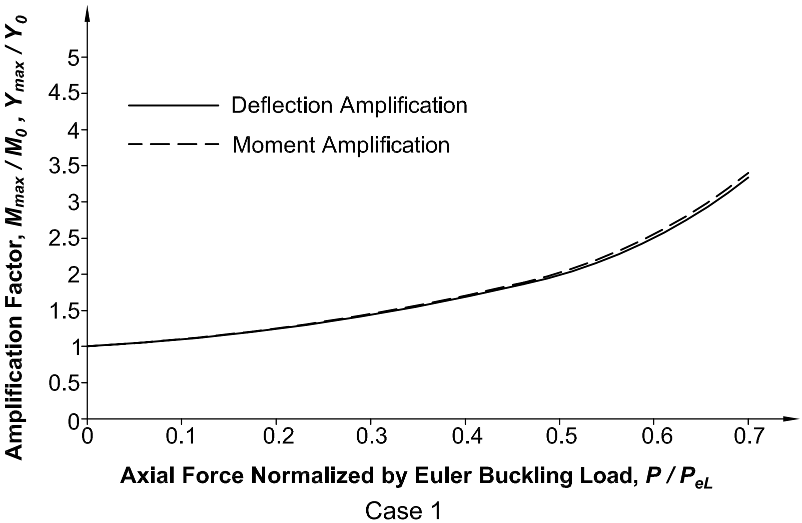
$$y_{MAX}\,(@x = L) = \frac{HL^3}{3EI}\left(\frac{3(\tan\alpha - \alpha)}{\alpha^3}\right) \quad \text{where } y_o = \frac{HL^3}{3EI}$$

Case 2

Fig. C-A-7.2. Benchmark problems.

amplification or B_2, the resulting errors in the internal forces are relatively small. If the notional loads are applied in combination with other lateral loads, there is no need for checking a B_2 limit. In all cases it is permitted to use the assumed out-of-plumbness geometry in the analysis of the structure in lieu of applying notional loads as an acceptable way to account for the geometric imperfection effects.

The third constraint (clauses 3 and 4) requires that the analysis be based on a reduced stiffness ($EI^* = 0.8\tau_b EI$ and $EA^* = 0.8EA$) in the structure. There are two reasons for imposing the reduced stiffness for analysis. First, for frames with slender members, where the limit state is governed by elastic stability, the



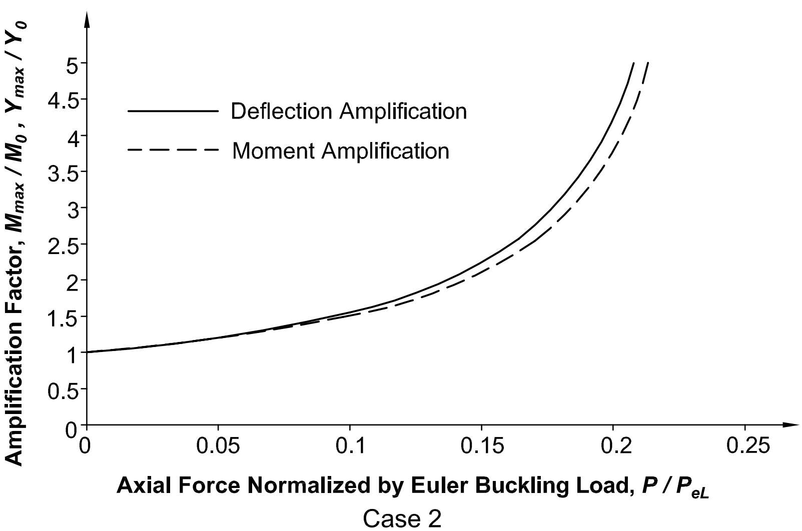


Fig. C-A-7.3. Maximum moment and deflection values as a function of axial force for benchmark problems.

0.8 factor on stiffness results in a system available strength equal to 0.8 times the elastic stability limit. This is roughly equivalent to the margin of safety implied by design of slender columns by the effective length procedure where the design strength $\phi P_n = 0.9(0.877)P_e = 0.79P_e$ where P_e is the elastic *critical load*, 0.90 is the specified resistance factor, and 0.877 is a reduction factor in the *column curve* equation (Equation E3-3). Second, for frames with intermediate or stocky columns, the $0.8\tau_b$ factor reduces the stiffness to account for inelastic softening prior to the members reaching their design strength. The τ_b factor is similar to the inelastic stiffness reduction factor implied in the *column curve* to account for loss of stiffness under high compression loads ($P_u > 0.5P_y$),and the 0.8 factor accounts for additional softening under combined axial compression and bending. It is a fortuitous coincidence that the reduction coefficients for both slender and stocky columns are close enough, such that the single reduction factor of $0.8\tau_b$works over the full range of slenderness. The reduced stiffness and notional load requirements only pertain to analyses for strength limit states. They do not apply to analyses of serviceability conditions of excessive deflections, vibration, etc. For ease of application in design practice, where $\tau_b = 1$, the reduction on *EI* and *EA* can be applied by modifying E in the analysis. However, for computer programs that do semi-automated design, one should ascertain that the reduced E is applied only for the second-order analysis. The elastic modulus should not be reduced in nominal strength equations that include E (for example, M_n for laterally unbraced beams). As shown in Figure C-A-7.1(b), the net effect of modifying the analysis in the manner just described is to amplify the second-order forces such that they are closer to the actual internal forces in the structure. It is for this reason that the beam-column interaction for in-plane flexural buckling is checked using an axial strength P_{nL} calculated from the *column curve* using the actual unbraced member length L, in other words, with $K = 1.0$.

In cases where the flexibility of other structural components (for example, connections, flexible column base details, or horizontal trusses acting as diaphragms) is modeled explicitly in the analysis, the stiffness of the other structural components should be reduced as well. Conservatively, the stiffness reduction may be taken as $EA^* = 0.8EA$ and/or $EI^* = 0.8EI$ for all cases. Surovek-Maleck, White, and Leon (2004) discuss the appropriate reduction of connection stiffnesses in the analysis of PR frames.

Simplified First-Order Analysis Based on the Direct Analysis Method (K = 1.0). The direct analysis method provides the technical basis for the provisions of Section C2.2b for design by first-order elastic analysis with $K = 1.0$ (Kuchenbecker, White, and Surovek-Maleck, 2004). The method is based on an assumed out-of-plumbness in the structure $\Delta_o/L = 0.002$, a target maximum drift ratio Δ/L, and reduced stiffnesses in the frame members ($0.8\tau_b EI$ and $0.8EA$). The first-order analysis is carried out using the nominal (unreduced) stiffness, and the above stiffness reduction is accounted for solely within the calculation of amplification factors. The method is applicable to braced, moment and combined frames. This method has a number of distinct advantages compared to the amplified first-order elastic approach specified in Chapter C:

(1) The second-order internal forces and moments are determined directly as part of the first-order analysis.
(2) There is no need to subdivide the analysis into artificial NT and LT parts.

Kuchenbecker and others (2004) present a general form of the suggested method. If the above approach is employed, it can be shown that for $B_2 \leq 1.5$ and $\tau_b = 1.0$ the required additional lateral load to be applied with other lateral loads in a first-order analysis of the structure, using the nominal (unreduced) stiffness, can be determined as:

$$N_i = \left(\frac{B_2}{1 - 0.2B_2}\right) \frac{\Delta}{\mathrm{L}} Y_i \geq \left(\frac{B_2}{1 - 0.2B_2}\right) 0.002Y_i \qquad \text{(C-A-7-3-1)}$$

where B_2 and Y_i are as defined in Chapter C, and Δ_H/L is the target maximum first-order drift ratio due to either the LRFD strength load combinations or 1.6 times the ASD strength load combinations. Note that if B_2 (based on the unreduced stiffness) is set to the 1.5 limit prescribed in Chapter C, then,

$$N_i = 2.1\,(\Delta/L)Y_i \geq 0.0042\;Y_i \qquad \text{(C-A-7-3-2)}$$

This is the additional lateral load required in Section C2.2b(2) of Chapter C.

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Specification for Structural Joints Using ASTM A325 or A490 Bolts

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Prepared by RCSC Committee A.1—Specifications and approved by the Research Council on Structural Connections.

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PREFACE

The purpose of the Research Council on Structural Connections (RCSC) is:

(1) To stimulate and support such investigation as may be deemed necessary and valuable to determine the suitability, strength and behavior of various types of structural connections;
(2) To promote the knowledge of economical and efficient practices relating to such structural connections; and,
(3) To prepare and publish related standards and such other documents as necessary to achieving its purpose.

The Council membership consists of qualified structural engineers from academic and research institutions, practicing design engineers, suppliers and manufacturers of fastener components, fabricators, erectors and code-writing authorities.

The first Specification approved by the Council, called the *Specification for Assembly of Structural Joints Using High Tensile Steel Bolts*, was published in January 1951. Since that time the Council has published fifteen successive editions. Each was developed through the deliberations and approval of the full Council membership and based upon past successful usage, advances in the state of knowledge and changes in engineering design practice. This edition of the Council's *Specification for Structural Joints Using ASTM A325 or A490 Bolts* continues the tradition of earlier editions. The major changes are:

- Sections 5.1, 5.2, and 5.3 were editorially revised to clarify strength requirements of slip critical connections.
- Section 6.2.1 was modified to permit the use of A490 type bolts, with round heads equal or larger in diameter than ASTM F1852 heads, without F436 hardened washers.
- Table 6.1, footnote d, was added to clarify use of non-hardened plate washer to be used in conjunction with an ASTM F436 hardened washer.
- Commentary Table C-2.1 bolt head and nut dimension locations F and W as shown in the artwork Figure C-2.2 was corrected.

In addition, typographical changes have been made throughout this Specification.

By the Research Council on Structural Connections,

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SYMBOLS

The following symbols are used in this Specification.

A_b Cross-sectional area based upon the nominal diameter of bolt, in.^{2}

D Slip probability factor as described in Section 5.4.2

D_u Multiplier that reflects the ratio of the mean installed bolt pretension to the specified minimum bolt pretension T_m as described in Section 5.4.1

F_n Nominal strength (per unit area), ksi

F_u Specified minimum tensile strength (per unit area), ksi

I Moment of inertia of the built-up member about the axis of buckling (see the Commentary to Section 5.4), in.^{4}

L Total length of the built-up member (see the Commentary to Section 5.4), in.

L_c Clear distance, in the direction of load, between the edge of the hole and the edge of the adjacent hole or the edge of the material, in.

N_b Number of bolts in the joint

P_u Required strength in compression, kips; Axial compressive force in the built-up member (see the Commentary to Section 5.4), kips

Q First moment of area of one component about the axis of buckling of the built-up member (see the Commentary to Section 5.4), in.^{3}

R_n Nominal strength, kips

R_s Service-load slip resistance, kips

T Applied service load in tension, kips

T_m Specified minimum bolt pretension (for pretensioned joints as specified in Table 8.1), kips

T_u Required strength in tension (factored tensile load), kips

V_u Required strength in shear (factored shear load), kips

d_b Nominal diameter of bolt, in.

t Thickness of the connected material, in.

t' Total thickness of fillers or shims (see Section 5.1), in.

k_s Slip coefficient for an individual specimen determined in accordance with Appendix A

ϕ Resistance factor

ϕR_n Design strength, kips

μ Mean slip coefficient

GLOSSARY

The following terms are used in this Specification. Where used, they are italicized to alert the user that the term is defined in this Glossary.

Coated Faying Surface. A *faying surface* that has been primed, primed and painted or protected against corrosion, except by hot-dip galvanizing.

Connection. An assembly of one or more *joints* that is used to transmit forces between two or more members.

Contractor. The party or parties responsible to provide, prepare and assemble the fastener components and connected parts described in this Specification.

Design Strength. ϕR_n, the resistance provided by an element or *connection*; the product of the *nominal strength* R_n and the resistance factor ϕ.

Engineer of Record. The party responsible for the design of the structure and for the approvals that are required in this Specification (see Section 1.4 and the corresponding Commentary).

Fastener Assembly. An assembly of fastener components that is supplied, tested and installed as a unit.

Faying Surface. The plane of contact between two plies of a *joint*.

Firm Contact. The condition that exists on a *faying surface* when the plies are solidly seated against each other, but not necessarily in continuous contact.

Galvanized Faying Surface. A *faying surface* that has been hot-dip galvanized.

Grip. The total thickness of the plies of a *joint* through which the bolt passes, exclusive of washers or direct-tension indicators.

Guide. The *Guide to Design Criteria for Bolted and Riveted Joints*, 2nd Edition (Kulak et al., 1987).

High-Strength Bolt. An ASTM A325 or A490 bolt, an ASTM F1852 twist-off-type tension-control bolt or an alternative-design fastener that meets the requirements in Section 2.8.

Inspector. The party responsible to ensure that the *contractor* has satisfied the provisions of this Specification in the work.

Joint. A bolted assembly with or without collateral materials that is used to join two structural elements.

Lot. In this Specification, the term *lot* shall be taken as that given in the ASTM Standard as follows:

Product	ASTM Standard	See Lot Definition in Section
Bolts	A325	9.4
	A490	11.3.2 or 11.4.2
Twist-off-type tension control bolt assemblies	F1852	13.4
Nuts	A563	9.2
Washers	F436	9.2
Compressible-washer-type direct tension indicators	F959	10.2.2

Manufacturer. The party or parties that produce the components of the *fastener assembly*.

Mean Slip Coefficient. μ, the ratio of the frictional shear load at the *faying surface* to the total normal force when slip occurs.

Nominal Strength. The capacity of a structure or component to resist the effects of loads, as determined by computations using the specified material strengths and dimensions and equations derived from accepted principles of structural mechanics or by field tests or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions.

Pretensioned Joint. A *joint* that transmits shear and/or tensile loads in which the bolts have been installed in accordance with Section 8.2 to provide a pretension in the installed bolt.

Protected Storage. The continuous protection of fastener components in closed containers in a protected shelter as described in the Commentary to Section 2.2.

Prying Action. Lever action that exists in *connections* in which the line of application of the applied load is eccentric to the axis of the bolt, causing deformation of the fitting and an amplification of the axial tension in the bolt.

Required Strength. The load effect acting on an element or *connection* determined by structural analysis from the factored loads using the most appropriate critical load combination.

Routine Observation. Periodic monitoring of the work in progress.

Shear/Bearing Joint. A *snug-tightened joint* or *pretensioned joint* with bolts that transmit shear loads and for which the design criteria are based upon the shear strength of the bolts and the bearing strength of the connected materials.

Slip-Critical Joint. A *joint* that transmits shear loads or shear loads in combination with tensile loads in which the bolts have been installed in accordance with Section 8.2 to provide a pretension in the installed bolt (clamping force on the *faying surfaces*), and with *faying surfaces* that have been prepared to provide a calculable resistance against slip.

Snug-Tightened Joint. A *joint* in which the bolts have been installed in accordance with Section 8.1. The snug-tightened condition is the tightness that is attained with a few impacts of an impact wrench or the full effort of an ironworker using an ordinary spud wrench to bring the plies into *firm contact*.

Start of Work. Any time prior to the installation of *high-strength bolts* in structural *connections* in accordance with Section 8.

Sufficient Thread Engagement. Having the end of the bolt extending beyond or at least flush with the outer face of the nut; a condition that develops the strength of the bolt.

Supplier. The party that sells the fastener components to the party that will install them in the work.

Tension Calibrator. A calibrated tension-indicating device that is used to verify the acceptability of the pretensioning method when a *pretensioned joint* or *slip-critical joint* is specified.

Uncoated Faying Surface. A *faying surface* that has neither been primed, painted, nor galvanized and is free of loose scale, dirt and other foreign material.

NOTES

SPECIFICATION FOR STRUCTURAL JOINTS USING ASTM A325 OR A490 BOLTS
June 30, 2004

SECTION 1. GENERAL REQUIREMENTS

1.1. Scope

This Specification covers the design of bolted *joints* and the installation and inspection of the assemblies of fastener components listed in Section 1.3, the use of alternative-design fasteners as permitted in Section 2.8 and alternative washer-type indicating devices as permitted in Section 2.6.2, in structural steel *joints*. This Specification relates only to those aspects of the connected materials that bear upon the performance of the fastener components. The Symbols, Glossary and Appendices are a part of this Specification.

Commentary:
This Specification deals principally with two strength grades of *high-strength bolts*, ASTM A325 and A490, and with their design, installation and inspection in structural steel *joints*. Equivalent fasteners, however, such as ASTM F1852 (equivalent to ASTM A325) twist-off-type tension-control bolt assemblies, are also covered. These provisions may not be relied upon for high-strength fasteners of other chemical composition, mechanical properties, or size. These provisions do not apply when material other than steel is included in the *grip*; nor are they applicable to anchor rods.

This Specification relates only to the performance of fasteners in structural steel *joints* and those few aspects of the connected material that affect this performance. Many other aspects of *connection* design and fabrication are of equal importance and must not be overlooked. For more general information on design and issues relating to *high-strength bolting* and the connected material, refer to current steel design textbooks and the *Guide to Design Criteria for Bolted and Riveted Joints*, 2nd Edition (Kulak et al., 1987).

1.2. Loads, Load Factors and Load Combinations

The design and construction of the structure shall conform to an applicable load and resistance factor design specification for steel structures. Because factored load combinations account for the reduced probabilities of maximum loads acting concurrently, the *design strengths* given in this Specification shall not be increased. Appendix B is included as an alternative approach.

Commentary:
This Specification is written in the load and resistance factor design (LRFD) format, which provides a method of proportioning structural components such that no applicable limit state is exceeded when the structure is subject to all appropriate load combinations. When a structure or structural component ceases to fulfill the intended purpose in some way, it is said to have exceeded a limit state. Strength limit states concern maximum load-carrying capability, and are related to

safety. Serviceability limit states are usually related to performance under normal service conditions, and usually are not related to strength or safety. The term "resistance" includes both strength limit states and serviceability limit states.

The *design strength* ϕR_n is the *nominal strength* R_n multiplied by the resistance factor ϕ. The factored load is the sum of the nominal loads multiplied by load factors, with due recognition of load combinations that account for the improbability of simultaneous occurrence of multiple transient load effects at their respective maximum values. The *design strength* ϕR_n of each structural component or assemblage must equal or exceed the *required strength* (V_u, T_u, etc.).

Although loads, load factors and load combinations are not explicitly specified in this Specification, the resistance factors herein are based upon those specified in ASCE 7. When the design is governed by other load criteria, the resistance factors specified herein should be adjusted as appropriate.

1.3. Referenced Standards and Specifications

The following standards and specifications are referenced herein:

American Institute of Steel Construction
Load and Resistance Factor Design Specification for Structural Steel Buildings, December 27,1999

American National Standards Institute
ANSI/ASME B18.2.6-96 *Fasteners for Use in Structural Applications*

American Society for Testing and Materials
ASTM A123-97a *Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products*

ASTM A153-98 *Standard Specification for Zinc Coating (Hot-Dip) on Iron and Steel Hardware*

ASTM A194-98b *Specification for Carbon and Alloy Steel Nuts for Bolts for High Pressure or High-Temperature Service, or Both*

ASTM A325-97 *Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength*

ASTM A490-97 *Standard Specification for Heat-Treated Steel Structural Bolts, 150 ksi Minimum Tensile Strength*

ASTM A563-97 *Standard Specification for Carbon and Alloy Steel Nuts*

ASTM B695-91[1] *Standard Specification for Coatings of Zinc Mechanically Deposited on Iron and Steel*

[1] Reapproved 1997.

ASTM F436-93 *Standard Specification for Hardened Steel Washers*

ASTM F959-99a *Standard Specification for Compressible-Washer-Type Direct Tension Indicators for Use with Structural Fasteners*

ASTM F1852-98 "*Twist off*" *Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength*

American Society of Civil Engineers
ASCE 7-98 *Minimum Design Loads for Buildings and Other Structures*

SSPC: The Society for Protective Coatings
SSPC-PA2-96 *Measurement of Dry Coating Thickness With Magnetic Gages*

Commentary:
Familiarity with the referenced AISC, ASCE, ASME, ASTM and SSPC specification requirements is necessary for the proper application of this Specification. The discussion of referenced specifications in this Commentary is limited to only a few frequently overlooked or misunderstood items.

1.4. Drawing Information

The *Engineer of Record* shall specify the following information in the contract documents

(1) The ASTM designation and type (Section 2) of bolt to be used;
(2) The *joint* type (Section 4);
(3) The required class of slip resistance if *slip-critical joints* are specified (Section 4); and,
(4) Whether slip is checked at the factored-load level or the service-load level, if *slip-critical joints* are specified (Section 5).

Commentary:
A summary of the information that the *Engineer of Record* is required to provide in the contract documents is provided in this Section. The parenthetical reference after each listed item indicates the location of the actual requirement in this Specification. In addition, the approval of the *Engineer of Record* is required in this Specification in the following cases:

(1) For the reuse of non-galvanized ASTM A325 bolts (Section 2.3.3);
(2) For the use of alternative washer-type indicating devices that differ from those that meet the requirements of ASTM F959, including the corresponding installation and inspection requirements that are provided by the *manufacturer* (Section 2.6.2);
(3) For the use of alternative-design fasteners, including the corresponding installation and inspection requirements that are provided by the *manufacturer* (Section 2.8);

(4) For the use of faying-surface coatings in *slip-critical joints* that provide a *mean slip coefficient* determined per Appendix A, but differing from Class A or Class B (Section 3.2.2(b));
(5) For the use of thermal cutting in the production of bolt holes (Section 3.3);
(6) For the use of oversized (Section 3.3.2), short-slotted (Section 3.3.3) or long slotted holes (Section 3.3.4) in lieu of standard holes;
(7) For the use of a value of D_u other than 1.13 (Section 5.4.1); and,
(8) For the use of a value of D other than 0.80 (Section 5.4.2).

SECTION 2. FASTENER COMPONENTS

2.1. Manufacturer Certification of Fastener Components

Manufacturer certifications documenting conformance to the applicable specifications required in Sections 2.3 through 2.8 for all fastener components used in the *fastener assemblies* shall be available to the *Engineer of Record* and *inspector* prior to assembly or erection of structural steel.

Commentary:
Certification by the *manufacturer* or *supplier* of *high-strength bolts*, nuts, washers and other components of the *fastener assembly* is required to ensure that the components to be used are identifiable and meet the requirements of the applicable ASTM Specifications.

2.2. Storage of Fastener Components

Fastener components shall be protected from dirt and moisture in closed containers at the site of installation. Only as many fastener components as are anticipated to be installed during the work shift shall be taken from *protected storage*. Fastener components that are not incorporated into the work shall be returned to *protected storage* at the end of the work shift. Fastener components shall not be cleaned or modified from the as-delivered condition.

Fastener components that accumulate rust or dirt shall not be incorporated into the work unless they are requalified as specified in Section 7. ASTM F1852 twist-off-type tension-control bolt assemblies and alternative-design fasteners that meet the requirements in Section 2.8 shall not be relubricated, except by the *manufacturer*.

Commentary:
Protected storage requirements are specified for *high-strength bolts*, nuts, washers and other fastener components with the intent that the condition of the components be maintained as nearly as possible to the as-manufactured condition until they are installed in the work. This involves:

(1) The storage of the fastener components in closed containers to protect from dirt and corrosion;
(2) The storage of the closed containers in a protected shelter;
(3) The removal of fastener components from *protected storage* only as necessary; and,
(4) The prompt return of unused fastener components to *protected storage*.

To facilitate manufacture, prevent corrosion and facilitate installation, the *manufacturer* may apply various coatings and oils that are present in the as manufactured condition. As such, the condition of supplied fastener components or the *fastener assembly* should not be altered to make them unsuitable for pre-tensioned installation.

If fastener components become dirty, rusty, or otherwise have their as received condition altered, they may be unsuitable for pre-tensioned installation.

It is also possible that a *fastener assembly* may not pass the pre-installation verification requirements of Section 7. Except for ASTM F1852 twist-off-type tension-control bolt assemblies (Section 2.7) and some alternative-design fasteners (Section 2.8), fastener components can be cleaned and lubricated by the fabricator or the erector. Because the acceptability of their installation is dependent upon specific lubrication, ASTM F1852 twist-off-type tension-control bolt assemblies and some alternative-design fasteners are suitable only if the *manufacturer* lubricates them.

2.3. Heavy-Hex Structural Bolts

2.3.1. Specifications: Heavy-hex structural bolts shall meet the requirements of ASTM A325 or ASTM A490. The *Engineer of Record* shall specify the ASTM designation and type of bolt (see Table 2.1) to be used.

2.3.2. Geometry: Heavy-hex structural bolt dimensions shall meet the requirements of ANSI/ASME B18.2.6. The bolt length used shall be such that the end of the bolt extends beyond or is at least flush with the outer face of the nut when properly installed.

Table 2.1. Acceptable ASTM A563 Nut Grade and Finish and ASTM F436 Washer Type and Finish

ASTM Desig.	Bolt Type	Bolt Finish[d]	ASTM A563 nut grade and finish[d]	ASTM F436 washer type and finish[a,d]
A325	1	Plain (uncoated)	C, C3, D, DH[c] and DH3; plain	1; plain
		Galvanized	DH[c]; galvanized And lubricated	1; galvanized
	3	Plain	C3 and DH3; plain	3; plain
F1852	1	Plain (uncoated)	C, C3, DH[c] and DH3; plain	1; plain[b]
		Mechanically Galvanized	DH[c]; mechanically galvanized and lubricated	1; mechanically galvanized[b]
	3	Plain	C3 and DH3; plain	3; plain[b]
A490	1	Plain	DH[c] and DH3; plain	1; plain
	3	Plain	DH3; plain	3; plain

[a] Applicable only if washer is required in Section 6.
[b] Required in all cases under nut per Section 6.
[c] The substitution of ASTM A194 grade 2H nuts in place of ASTM A563 grade DH nuts is permitted.
[d] "Galvanized" as used in this table refers to hot-dip galvanizing in accordance with ASTM A153 or mechanical galvanizing in accordance with ASTM B695.

2.3.3. Reuse: ASTM A490 bolts and galvanized ASTM A325 bolts shall not be reused. When approved by the *Engineer of Record*, black ASTM A325 bolts are permitted to be reused. Touching up or re-tightening bolts that may have been loosened by the installation of adjacent bolts shall not be considered to be a reuse.

Commentary:
ASTM A325 and ASTM A490 currently provide for two types (according to metallurgical classification) of *high-strength bolts*, supplied in diameters from ½ in. to 1½ in. inclusive. Type 1 covers medium carbon steel for ASTM A325 bolts and alloy steel for ASTM A490 bolts. Type 3 covers *high-strength bolts* that have improved atmospheric corrosion resistance and weathering characteristics. (Reference to Type 2 ASTM A325 and Type 2 A490 bolts, which appeared in previous editions of this Specification, has been removed following the removal of similar reference within the ASTM A325 and A490 Specifications). When the bolt type is not specified, either Type 1 or Type 3 may be supplied at the option of the *manufacturer*. Note that ASTM F1852 twist-off-type tension-control bolt assemblies may be manufactured with a button head or hexagonal head; other requirements for these *fastener assemblies* are found in Section 2.7.

Regular heavy-hex structural bolts and twist-off-type tension-control bolt assemblies are required by ASTM Specifications to be distinctively marked. Certain markings are mandatory. In addition to the mandatory markings, the *manufacturer* may apply additional distinguishing markings. The mandatory and sample optional markings are illustrated in Figure C-2.1.

ASTM Specifications permit the galvanizing of ASTM A325 bolts but not ASTM A490 bolts. Similarly, the application of zinc to ASTM A490 bolts by metallizing or mechanical coating is not permitted because the effect of mechanical galvanizing on embrittlement and delayed cracking of ASTM A490 bolts has not been fully investigated to date.

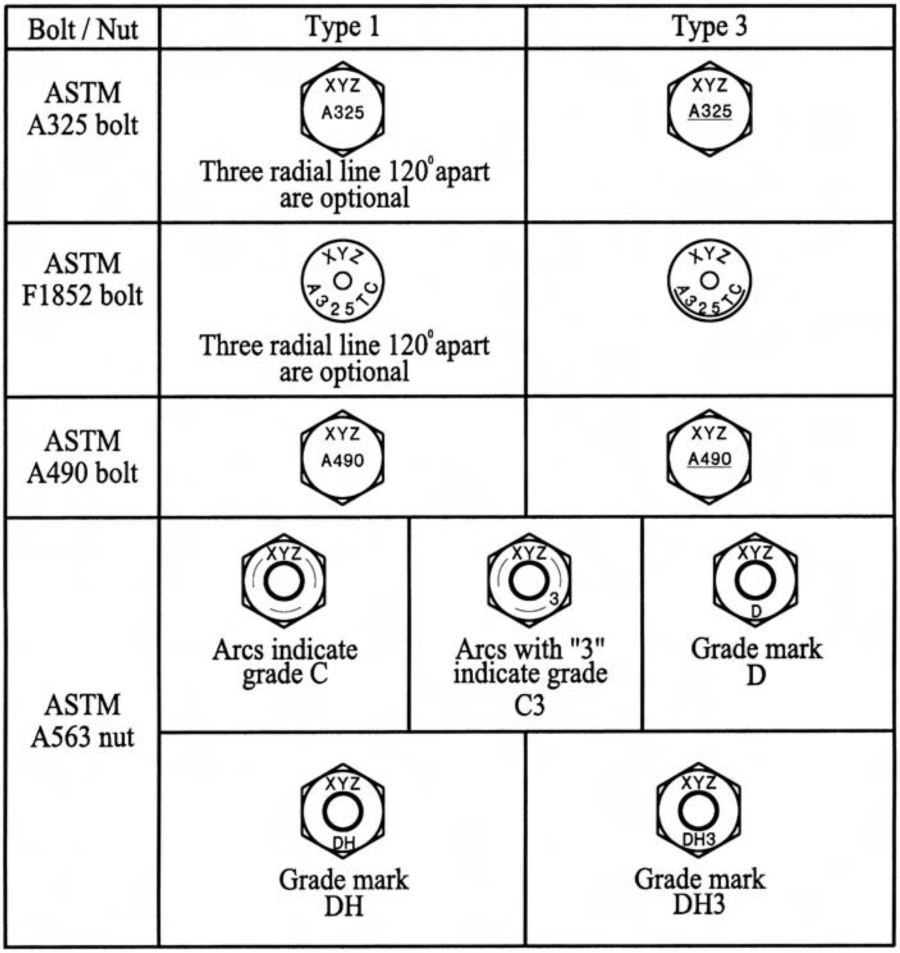
Galvanized *high-strength bolts* and nuts must be considered as a manufactured *fastener assembly*. Insofar as the hot-dip galvanized bolt and nut assembly is concerned, four principal factors must be considered so that the provisions of this Specification are understood and properly applied. These are:

(1) The effect of the hot-dip galvanizing process on the mechanical properties of high-strength steels;
(2) The effect of over-tapping for hot-dip galvanized coatings on the nut stripping strength;
(3) The effect of galvanizing and lubrication on the torque required for pretensioning; and,
(4) Shipping requirements.

Birkemoe and Herrschaft (1970) showed that, in the as-galvanized condition, galvanizing increases the friction between the bolt and nut threads as well as the variability of the torque-induced pretension. A lower required torque and more consistent results are obtained if the nuts are lubricated. Thus, it is required in ASTM A325 that a galvanized bolt and lubricated galvanized nut be assembled in a steel *joint* with a galvanized washer and tested by the *supplier* prior to shipment. This testing must show that the galvanized nut with the lubricant provided may be rotated from the snug-tight condition well in excess of the rotation required for pretensioned installation without stripping. This requirement applies to both hot-dip and mechanically galvanized fasteners. The above requirements clearly indicate that:

(1) Galvanized *high-strength bolts* and nuts must be treated as a *fastener assembly*;

(2) The *supplier* must supply nuts that have been lubricated and tested with the supplied *high-strength bolts*;



Notes:
1. XYZ represents the manufacturer's identification mark.
2. ASTM F1852 twist-off-type tension-control bolt assemblies are also produced with heavy-hex head that has similar markings.

Figure C-2.1. Required marks for acceptable bolt and nut assemblies.

(3) Nuts and *high-strength bolts* must be shipped together in the same shipping container; and,

(4) The purchase of galvanized *high-strength bolts* and galvanized nuts from separate *suppliers* is not in accordance with the intent of the ASTM Specifications because the control of over-tapping, the testing and application of lubricant and the *supplier* responsibility for the performance of the assembly would clearly not have been provided as required.

Because some of the lubricants used to meet the requirements of ASTM Specifications are water soluble, it is advisable that galvanized *high-strength bolts* and nuts be shipped and stored in plastic bags or in sealed wood or metal containers. Containers of fasteners with hot-wax-type lubricants should not be subjected to heat that would cause depletion or change in the properties of the lubricant.

Both the hot-dip galvanizing process (ASTM A153) and the mechanical galvanizing process (ASTM B695) are recognized in ASTM A325. The effects of the two processes upon the performance characteristics and requirements for proper installation are distinctly different. Therefore, distinction between the two must be noted in the comments that follow. In accordance with ASTM A325, all threaded components of the *fastener assembly* must be galvanized by the same process and the *supplier's* option is limited to one process per item with no mixed processes in a *lot*. Mixing *high-strength bolts* that are galvanized by one process with nuts that are galvanized by the other may result in an unworkable assembly.

Steels in the 200 ksi and higher tensile-strength range are subject to embrittlement if hydrogen is permitted to remain in the steel and the steel is subjected to high tensile stress. The minimum tensile strength of ASTM A325 bolts is 105 ksi or 120 ksi, depending upon the diameter, and maximum hardness limits result in production tensile strengths well below the critical range. The maximum tensile strength for ASTM A490 bolts was set at 170 ksi to provide a little more than a ten-percent margin below 200 ksi. However, because *manufacturers* must target their production slightly higher than the required minimum, ASTM A490 bolts close to the critical range of tensile strength must be anticipated. For black *high-strength bolts*, this is not a cause for concern. However, if the bolt is hot-dip galvanized, delayed brittle fracture in service is a concern because of the possibility of the introduction of hydrogen during the pickling operation of the hot-dip galvanizing process and the subsequent "sealing-in" of the hydrogen by the zinc coating. There also exists the possibility of cathodic hydrogen absorption arising from the corrosion process in certain aggressive environments.

ASTM A325 and A490 bolts are manufactured to dimensions as specified in ANSI/ASME B18.2.6. The basic dimensions, as defined in Figure C-2.2, are shown in Table C-2.1.

The principal geometric features of heavy-hex structural bolts that distinguish them from bolts for general application are the size of the head and the unthreaded body length. The head of the heavy-hex structural bolt is specified to be the same size as a heavy-hex nut of the same nominal diameter so that the ironworker may use the same wrench or socket either on the bolt head and/or on the nut. With the specific exception of fully threaded ASTM A325T bolts as discussed below, heavy-hex structural bolts have shorter threaded lengths than bolts for general applications. By making the body length of the bolt the control dimension, it has been possible to exclude the thread from all shear planes when desirable, except for the case of thin outside parts adjacent to the nut.

The shorter threaded lengths provided with heavy-hex structural bolts tend to minimize the threaded portion of the bolt within the *grip*. Accordingly, care must also be exercised to provide adequate threaded length between the nut and the bolt head to enable appropriate installation without jamming the nut on the thread run-out.

Depending upon the increments of supplied bolt lengths, the full thread may extend into the *grip* for an assembly without washers by as much as 3/8 in. for 1/2, 5/8, 3/4, 7/8, 1 1/4, and 1 1/2 in. diameter *high-strength bolts* and as much as 1/2 in. for 1,

Table C-2.1. Bolt and Nut Dimensions

Nominal Bolt Diameter d_b, in.	Heavy Hex Structural Bolt Dimensions			Heavy Hex Nut Dimensions	
	Width across flats F, in.	Height H_1, in.	Thread Length T, in.	Width across flats W, in.	Height H_2, in.
1/2	7/8	5/16	1	7/8	31/64
5/8	1 1/16	25/64	1 1/4	1 1/16	39/64
3/4	1 1/4	15/32	1 3/8	1 1/4	47/64
7/8	1 7/16	35/64	1 1/2	1 7/16	55/64
1	1 5/8	39/64	1 3/4	1 5/8	63/64
1 1/8	1 13/16	11/16	2	1 13/16	1 7/64
1 1/4	2	25/32	2	2	1 7/32
1 3/8	2 3/16	27/32	2 1/4	2 3/16	1 11/32
1 1/2	2 3/8	15/16	2 1/4	2 3/8	1 15/32

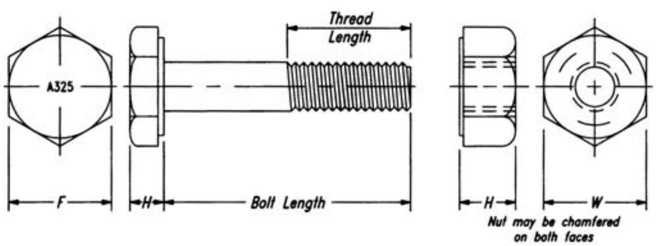


Figure C-2.2. Heavy-hex structural bolt and heavy-hex nut.

1⅛, and 1⅜ in. diameter *high-strength bolts*. When the thickness of the ply closest to the nut is less than the ⅜ in. or ½ in. dimensions given above, it may still be possible to exclude the threads from the shear plane, when required, depending upon the specific combination of bolt length, *grip* and number of washers used under the nut (Carter, 1996). If necessary, the next increment of bolt length can be specified with ASTM F436 washers in sufficient number to both exclude the threads from the shear plane and ensure that the assembly can be installed with adequate threads included in the *grip* for proper installation.

At maximum accumulation of tolerances from all components in the *fastener assembly*, the thread run-out will cross the shear plane for the critical combination of bolt length and *grip* used to select the foregoing rules of thumb for ply thickness required to exclude the threads. This condition is not of concern, however, for two reasons. First, it is too unlikely that all component tolerances will accumulate at their maximum values to warrant consideration. Second, even if the maximum accumulation were to occur, the small reduction in shear strength due to the presence of the thread run-out (not a full thread) would be negligible.

There is an exception to the foregoing thread length requirements for ASTM A325 bolts but not for ASTM A490 bolts nor ASTM F1852 twist-off-type tension-control bolt assemblies. Supplementary requirements in ASTM A325 permit the purchaser to specify a bolt that is threaded for the full length of the shank, when the bolt length is equal to or less than four times the nominal diameter. This exception is provided to increase economy through simplified ordering and inventory control in the fabrication and erection of some structures. It is particularly useful in those structures in which the strength of the *connection* is dependent upon the bearing strength of relatively thin connected material rather than the shear strength of the bolt, whether with threads in the shear plane or not. As required in ASTM A325, *high-strength bolts* ordered to such supplementary requirements must be marked with the symbol A325T.

To determine the required bolt length, the value shown in Table C-2.2 should be added to the *grip* (i.e., the total thickness of all connected material, exclusive of washers). For each ASTM F436 washer that is used, add 5⁄32 in.; for each beveled washer, add 5⁄16 in. The tabulated values provide appropriate allowances for manufacturing tolerances and also provide *sufficient thread engagement* with an installed heavy-hex nut. The length determined by the use of Table C-2.2 should be adjusted to the next longer ¼-in. length increment (½-in. length increment for lengths exceeding 6 in.). A more extensive table for bolt length selection based upon these rules is available (Carter, 1996).

Pretensioned installation involves the inelastic elongation of the portion of the threaded length between the nut and the thread run-out. ASTM A490 bolts and galvanized ASTM A325 bolts possess sufficient ductility to undergo one pretensioned installation, but are not consistently ductile enough to undergo a second pretensioned installation. Black ASTM A325 bolts, however, possess sufficient ductility to undergo more than one pretensioned installation as suggested in the *Guide* (Kulak et al., 1987). As a simple rule of thumb, a black ASTM A325 bolt is suitable for reuse if the nut can be run up the threads by hand.

Table C- 2.2. Bolt Length Selection Increment

Nominal Bolt Diameter d_b, in.	To Determine the Required Bolt Length, Add to Grip, in.
$\frac{1}{2}$	$\frac{11}{16}$
$\frac{5}{8}$	$\frac{7}{8}$
$\frac{3}{4}$	1
$\frac{7}{8}$	$1\frac{1}{8}$
1	$1\frac{1}{4}$
$1\frac{1}{8}$	$1\frac{1}{2}$
$1\frac{1}{4}$	$1\frac{5}{8}$
$1\frac{3}{8}$	$1\frac{3}{4}$
$1\frac{1}{2}$	$1\frac{7}{8}$

2.4. Heavy-Hex Nuts

2.4.1. Specifications: Heavy-hex nuts shall meet the requirements of ASTM A563. The grade and finish of such nuts shall be as given in Table 2.1.

2.4.2. Geometry: Heavy-hex nut dimensions shall meet the requirements of ANSI/ASME B18.2.6.

Commentary:
Heavy-hex nuts are required by ASTM Specifications to be distinctively marked. Certain markings are mandatory. In addition to the mandatory markings, the *manufacturer* may apply additional distinguishing markings. The mandatory markings and sample optional markings are illustrated in Figure C-2.1.

Hot-dip galvanizing affects the stripping strength of the bolt-nut assembly because, to accommodate the relatively thick zinc coatings of non-uniform thickness on bolt threads, it is usual practice to hot-dip galvanize the blank nut and then to tap the nut over-size. This results in a reduction of thread engagement with a consequent reduction of the stripping strength. Only the stronger hardened nuts have adequate strength to meet ASTM thread strength requirements after over-tapping. Therefore, as specified in ASTM A325, only ASTM A563 grade DH are suitable for use as galvanized nuts. This requirement should not be overlooked if non-galvanized nuts are purchased and then sent to a local galvanizer for hot-dip galvanizing. Because the mechanical galvanizing process results in a more uniformly distributed and smooth zinc coating, nuts may be tapped over-size before galvanizing by an amount that is less than that required for the hot-dip process before galvanizing.

In earlier editions, this Specification permitted the use of ASTM A194 grade 2H nuts in the same finish as that permitted for ASTM A563 nuts in the following cases: with ASTM A325 Type 1 plain, Type 1 galvanized and Type 3 plain bolts and with ASTM A490 Type 1 plain bolts. Reference to ASTM A194 grade 2H nuts has been removed following the removal of similar reference within the ASTM A325 and A490 Specifications. However, it should be noted that ASTM A194 grade 2H nuts remain acceptable in these applications as indicated by footnote in Table 2.1, should they be available.

ASTM A563 nuts are manufactured to dimensions as specified in ANSI/ASME B18.2.6. The basic dimensions, as defined in Figure C-2.2, are shown in Table C-2.1.

2.5. Washers

Flat circular washers and square or rectangular beveled washers shall meet the requirements of ASTM F436, except as provided in Table 6.1. The type and finish of such washers shall be as given in Table 2.1.

2.6. Washer-Type Indicating Devices

The use of washer-type indicating devices is permitted as described in Sections 2.6.1 and 2.6.2.

2.6.1. Compressible-Washer-Type Direct Tension Indicators: Compressible-washer-type direct tension indicators shall meet the requirements of ASTM F959.

2.6.2. Alternative Washer-Type Indicating Devices: When approved by the *Engineer of Record*, the use of alternative washer-type indicating devices that differ from those that meet the requirements of ASTM F959 is permitted.

Detailed installation instructions shall be prepared by the *manufacturer* in a supplemental specification that is approved by the *Engineer of Record* and shall provide for:

(1) The required character and frequency of pre-installation verification;
(2) The alignment of bolt holes to permit insertion of the bolt without undue damage to the threads;
(3) The placement of *fastener assemblies* in all types and sizes of holes, including placement and orientation of the alternative and regular washers;
(4) The systematic assembly of the *joint*, progressing from the most rigid part of the *joint* until the connected plies are in *firm contact*; and;
(5) The subsequent systematic pretensioning of all bolts in the *joint*, progressing from the most rigid part of the *joint* in a manner that will minimize relaxation of previously pretensioned bolts.

Detailed inspection instructions shall be prepared by the *manufacturer* in a supplemental specification that is approved by the *Engineer of Record* and shall provide for:

(1) Observation of the required pre-installation verification testing; and,
(2) Subsequent *routine observation* to ensure the proper use of the alternative washer-type indicating device.

2.7. Twist-Off-Type Tension-Control Bolt Assemblies

2.7.1. Specifications: Twist-off-type tension-control bolt assemblies shall meet the requirements of ASTM F1852. The *Engineer of Record* shall specify the type of bolt (Table 2.1) to be used.

2.7.2. Geometry: Twist-off-type tension-control bolt assembly dimensions shall meet the requirements of ASTM F1852. The bolt length used shall be such that the end of the bolt extends beyond or is at least flush with the outer face of the nut when properly installed.

Commentary:
It is the policy of the Research Council on Structural Connections to directly recognize only those fastener components that are manufactured to meet the requirements in an approved ASTM specification. Prior to this edition, the RCSC Specification provided for the use of ASTM A325 and A490 bolts directly and alternative-design fasteners meeting detailed requirements similar to those in Section 2.8 when approved by the *Engineer of Record*. With this edition, ASTM F1852 twist-off-type tension-control bolt assemblies are now recognized directly. Essentially, ASTM F1852 relates an ASTM A325-equivalent product to a specific method of installation that is suitable for use in all *joint* types as described in Section 8. Provision has also been retained for approval by the *Engineer of Record* of other alternative-design fasteners that meet the detailed requirements in 2.8. As an example of one such approval, the use of twist-off-type tension-control bolt assemblies with ASTM A490 mechanical properties is usually deemed acceptable.

If galvanized, ASTM F1852 twist-off-type tension-control bolt assemblies are required in ASTM F1852 to be mechanically galvanized.

While specific provisions for reuse of ASTM F1852 twist-off-type tension control bolts have not been included in this Specification, those given in Section 2.3.3 for reuse of heavy-hex structural bolts are equally applicable if the use of an alternative pretensioning method, such as the turn-of-nut pretensioning method, is practical. It is assumed that rotation of the non-turned element can be restrained.

2.8. Alternative-Design Fasteners

When approved by the *Engineer of Record*, the use of alternative-design fasteners is permitted if they:

(1) Meet the materials, manufacturing and chemical composition requirements of ASTM A325 or ASTM A490, as applicable;
(2) Meet the mechanical property requirements of ASTM A325 or ASTM A490 in full-size tests;
(3) Have a body diameter and bearing area under the bolt head and nut that is equal to or greater than those provided by a bolt and nut of the same nomi-

nal dimensions specified in Sections 2.3 and 2.4; and,

(4) Are supplied and used in the work as a *fastener assembly*.

Such alternative-design fasteners are permitted to differ in other dimensions from those of the specified *high-strength bolts* and nuts.

Detailed installation instructions shall be prepared by the *manufacturer* in a supplemental specification that is approved by the *Engineer of Record* and shall provide for:

(1) The required character and frequency of pre-installation verification;
(2) The alignment of bolt holes to permit insertion of the alternative-design fastener without undue damage;
(3) The placement of *fastener assemblies* in all holes, including any washer requirements as appropriate;
(4) The systematic assembly of the *joint*, progressing from the most rigid part of the *joint* until the connected plies are in *firm contact*; and,
(5) The subsequent systematic pretensioning of all *fastener assemblies* in the *joint*, progressing from the most rigid part of the *joint* in a manner that will minimize relaxation of previously pretensioned bolts.

Detailed inspection instructions shall be prepared by the *manufacturer* in a supplemental specification that is approved by the *Engineer of Record* and shall provide for:

(1) Observation of the required pre-installation verification testing; and,
(2) Subsequent *routine observation* to ensure the proper use of the alternative-design fastener.

SECTION 3. BOLTED PARTS

3.1. Connected Plies

All connected plies that are within the *grip* of the bolt and any materials that are used under the head or nut shall be steel (uncoated, coated or galvanized) as defined in Section 3.2. Compressible materials shall not be placed within the *grip* of the bolt. The slope of the surfaces of parts in contact with the bolt head and nut shall be equal to or less than 1:20 with respect to a plane that is normal to the bolt axis.

Commentary:

The presence of gaskets, insulation or any compressible materials other than the specified coatings within the *grip* would preclude the development and/or retention of the installed pretensions in the bolts, when required.

ASTM A325, F1852 and A490 bolt assemblies are ductile enough to deform to a surface with a slope that is less than or equal to 1:20 with respect to a plane normal to the bolt axis. Greater slopes are undesirable because the resultant localized bending decreases both the strength and the ductility of the bolt.

3.2. Faying Surfaces

Faying surfaces and surfaces adjacent to the bolt head and nut shall be free of dirt and other foreign material. Additionally, *faying surfaces* shall meet the requirements in Sections 3.2.1 or 3.2.2.

3.2.1. *Snug-Tightened Joints* and *Pretensioned Joints*: The *faying surfaces* of *snug-tightened joints* and *pretensioned joints* as defined in Sections 4.1 and 4.2 are permitted to be uncoated, coated with coatings of any formulation or galvanized.

Commentary:

In both *snug-tightened joints* and *pretensioned joints*, the ultimate strength is dependent upon shear transmitted by the bolts and bearing of the bolts against the connected material. It is independent of any frictional resistance that may exist on the *faying surfaces*. Consequently, since slip resistance is not an issue, the *faying surfaces* are permitted to be uncoated, coated, or galvanized without regard to the resulting slip coefficient obtained.

3.2.2. *Slip-Critical Joints*: The *faying surfaces* of *slip-critical joints* as defined in Section 4.3, including those of filler plates and finger shims, shall meet the following requirements:

(a) *Uncoated Faying Surfaces*: *Uncoated faying surfaces* shall be free of scale, except tight mill scale, and free of coatings, including inadvertent overspray, in areas closer than one bolt diameter but not less than 1 in. from the edge of any hole and in all areas within the bolt pattern.

(b) *Coated Faying Surfaces*: *Coated faying surfaces* shall first be blast cleaned and subsequently coated with a coating that is qualified in accordance with the requirements in Appendix A as a Class A or Class B coating as defined in Section 5.4. Alternatively, when approved by the *Engineer of Record*, coatings that provide a *mean slip coefficient* that differs from Class A or Class B are permitted when:

(1) The *mean slip coefficient* μ is established by testing in accordance with the requirements in Appendix A; and,
(2) The design slip resistance is determined in accordance with Section 5.4 using this coefficient, except that, for design purposes, a value of μ greater than 0.50 shall not be used.

The plies of *slip-critical joints* with *coated faying surfaces* shall not be assembled before the coating has cured for the minimum time that was used in the qualifying tests.

(c) *Galvanized Faying Surfaces*: *Galvanized faying surfaces* shall first be hot-dip galvanized in accordance with the requirements of ASTM A123 and subsequently roughened by means of hand wire brushing. Power wire brushing is not permitted. When prepared by roughening, the *galvanized faying surface* is designated as Class C for design.

Commentary:
Slip-critical joints are those *joints* that have specified *faying surface* conditions that, in the presence of the clamping force provided by pretensioned fasteners, resist a design load solely by friction and without displacement at the *faying surfaces*. Consequently, it is necessary to prepare the *faying surfaces* in a manner so that the desired slip performance is achieved.

Clean mill scale steel surfaces (Class A, see Section 5.4.1) and blast-cleaned steel surfaces (Class B, see Section 5.4.1) can be used within *slip-critical joints*. When used, it is necessary to keep the *faying surfaces* free of coatings, including inadvertent overspray.

Corrosion often occurs on uncoated blast-cleaned steel surfaces (Class B, see Section 5.4.1) due to exposure between the time of fabrication and subsequent erection. In normal atmospheric exposures, this corrosion is not detrimental and may actually increase the slip resistance of the *joint*. Yura et al. (1981) found that the Class B slip coefficient could be maintained for up to one year prior to *joint* assembly.

Polyzois and Frank (1986) demonstrated that, for plate material with thickness in the range of ⅜ in. to ¾ in., the contact pressure caused by bolt pretension is concentrated on the *faying surfaces* in annular rings around and close to the bolts. In this study, unqualified paint on the *faying surfaces* away from the edge of the bolt hole by not less than 1 in. nor the bolt diameter did not reduce the slip resistance. However, this would not likely be the case for *joints* involving thicker material, particularly those with a large number of bolts on multiple gage lines; the Table 8.1 minimum bolt pretension might not be adequate to completely flatten and pull thicker material into tight contact around every bolt. Instead, the bolt pretension would be balanced by contact pressure on the regions of the *faying surfaces* that are in contact. To account for both possibilities, it is required in this Specification that all areas between the bolts be free of coatings, including overspray, as illustrated in Figure C-3.1.

As a practical matter, the smaller coating-free area can be laid out and protected more easily using masking located relative to the bolt-hole pattern than relative to the limits of the complete area of *faying surface* contact with varying and uncertain edge distance. Furthermore, the narrow coating strip around the perimeter of the *faying surface* minimizes the required field touch-up of uncoated material outside of the *joint*.

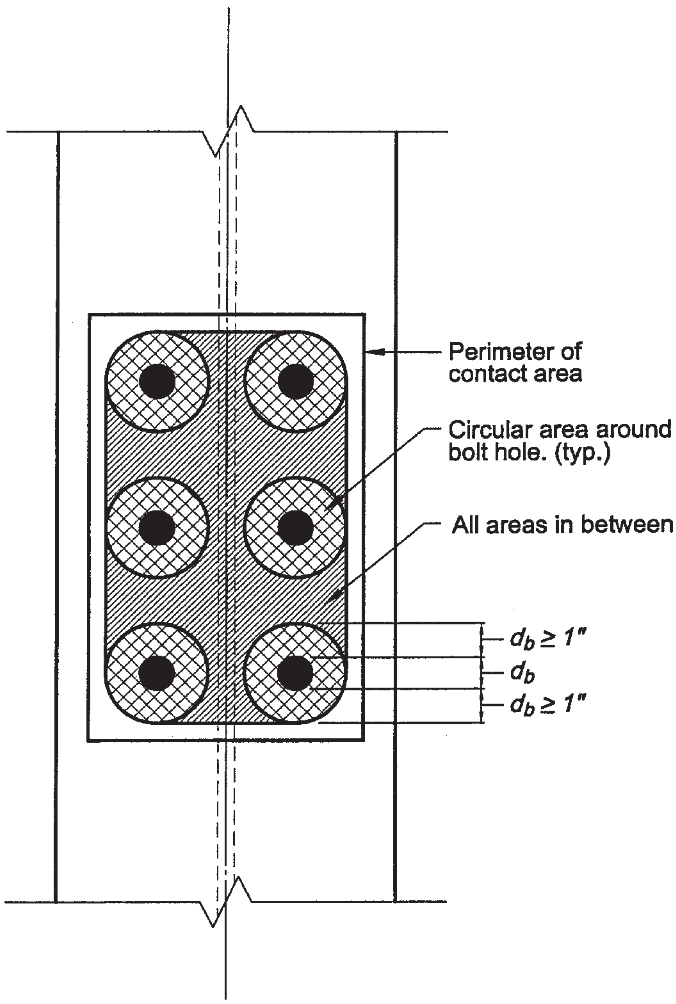


Figure C-3.1. Faying surfaces of slip-critical connections painted with unqualified paints.

Polyzois and Frank (1986) also investigated the effect of various degrees of inadvertent overspray on slip resistance. It was found that even a small amount of overspray of unqualified paint (that is, not qualified as a Class A or Class B coating) within the specified coating-free area on clean mill scale can reduce the slip resistance significantly. On blast-cleaned surfaces, however, the presence of a small amount of overspray was not as detrimental. For simplicity, this Specification requires that all overspray be prohibited from areas that are required to be free of coatings in *slip-critical joints* regardless of whether the surface is clean mill scale steel or blast-cleaned steel.

In the 1980 edition of this Specification, generic names for coatings applied to *faying surfaces* were the basis for categories of allowable working stresses in *slip-critical* (friction) *joints*. Frank and Yura (1981) demonstrated that the slip coefficients for coatings described by a generic type are not unique values for a given generic coating description or product, but rather depend also upon the type of vehicle used. Small differences in formulation from *manufacturer* to *manufacturer* or from *lot* to *lot* with a single *manufacturer* can significantly affect slip coefficients if certain essential variables within a generic type are changed. Consequently, it is unrealistic to assign coatings to categories with relatively small incremental differences between categories based solely upon a generic description.

When the *faying surfaces* of a *slip-critical joint* are to be protected against corrosion, a qualified coating must be used. A qualified coating is one that has been tested in accordance with Appendix A, the sole basis for qualification of any coating to be used in conjunction with this Specification. Coatings can be qualified as follows:

(1) As a Class A coating as defined in Section 5.4.1;
(2) As a Class B coating as defined in Section 5.4.1; or,
(3) As a coating with a *mean slip coefficient* μ other than 0.33 (Class A) but not greater than 0.50 (Class B).

Requalification is required if any essential variable associated with surface preparation, paint manufacture, application method or curing requirements is changed. See Appendix A.

Frank and Yura (1981) also investigated the effect of varying the time between coating the *faying surfaces* and assembly of the *joint* and pretensioning the bolts in order to ascertain if partially cured paint continued to cure within the assembled *joint* over a period of time. The results indicated that all curing effectively ceased at the time the *joint* was assembled and paint that was not fully cured at that time acted as a lubricant. The slip resistance of a *joint* that was assembled after a time less than the curing time used in the qualifying tests was severely reduced. Thus, the curing time prior to mating the *faying surfaces* is an essential parameter to be specified and controlled during construction.

The *mean slip coefficient* for clean hot-dip galvanized surfaces is on the order of 0.19 as compared with a factor of about 0.33 for clean mill scale. Birkemoe and Herrschaft (1970) showed that this *mean slip coefficient* can be significantly improved by treatments such as hand wire brushing or light "brush-off" grit blasting. In either case, the treatment must be controlled to achieve visible roughening or scoring. Power wire brushing is unsatisfactory because it may polish rather than roughen the surface, or remove the coating.

Field experience and test results have indicated that galvanized assemblies may continue to slip under sustained loading (Kulak et al., 1987; pp. 198-208). Tests of hot-dip galvanized *joints* subjected to sustained loading show a creep-type behavior that was not observed in short-duration or fatigue-type load application. See also the Commentary to Appendix A.

3.3. Bolt Holes

The nominal dimensions of standard, oversized, short-slotted and long-slotted holes for *high-strength bolts* shall be equal to or less than those shown in Table 3.1. Thermally cut bolt holes shall be permitted if approved by the *Engineer of Record*. For statically loaded *joints*, thermally cut surfaces need not be ground. For cyclically loaded *joints*, thermally cut surfaces shall be ground smooth.

Commentary:

The footnotes in Table 3.1 provide for slight variations in the dimensions of bolt holes from the nominal dimensions. When the dimensions of bolt holes are such that they exceed these permitted variations, the bolt hole must be treated as the next larger type.

3.3.1. Standard Holes: In the absence of approval by the *Engineer of Record* for the use of other hole types, standard holes shall be used in all plies of bolted *joints*.

Table 3.1. Nominal Bolt Hole Dimensions

Nominal Bolt Diameter, d_b, in.	Nominal Bolt Hole Dimensions[a,b], in.			
	Standard (diameter)	**Oversized (diameter)**	**Short-slotted (width × length)**	**Long-slotted (width × length)**
1/2	9/16	5/8	9/16 × 11/16	9/16 × 1 1/4
5/8	11/16	13/16	11/16 × 7/8	11/16 × 1 9/16
3/4	13/16	15/16	13/16 × 1	13/16 × 1 7/8
7/8	15/16	1 1/16	15/16 × 1 1/8	15/16 × 2 3/16
1	1 1/16	1 1/4	1 1/16 × 1 5/16	1 1/16 × 2 1/2
≥ 1 1/8	d_b + 1/16	d_b + 5/16	(d_b + 1/16) × (d_b + 3/8)	(d_b + 1/16) × (2.5d_b)

[a] The upper tolerance on the tabulated nominal dimensions shall not exceed 1/32-in. Exception: In the width of slotted holes, gouges not more than 1/16-in. deep are permitted.

[b] The slightly conical hole that naturally results from punching operations with properly matched punches and dies is acceptable.

Commentary:
The use of bolt holes $\frac{1}{16}$ in. larger than the bolt installed in them has been permitted since the first publication of this Specification. Allen and Fisher (1968) showed that larger holes could be permitted for *high-strength bolts* without adversely affecting the bolt shear or member bearing strength. However, the slip resistance can be reduced by the failure to achieve adequate pretension initially or by the relaxation of the bolt pretension as the highly compressed material yields at the edge of the hole or slot. The provisions for oversized and slotted holes in this Specification are based upon these findings and the additional concern for the consequences of a slip of significant magnitude if it should occur in the direction of the slot. Because an increase in hole size generally reduces the net area of a connected part, the use of oversized holes or of slotted holes is subject to approval by the *Engineer of Record*.

3.3.2. Oversized Holes: When approved by the *Engineer of Record*, oversized holes are permitted in any or all plies of *slip-critical joints* as defined in Section 4.3.

Commentary:
See the Commentary to Section 3.3.1.

3.3.3. Short-Slotted Holes: When approved by the *Engineer of Record*, short-slotted holes are permitted in any or all plies of *snug-tightened joints* as defined in Section 4.1, and *pretensioned joints* as defined in Section 4.2, provided the applied load is approximately perpendicular (between 80 and 100 degrees) to the axis of the slot. When approved by the *Engineer of Record*, short-slotted holes are permitted in any or all plies of *slip-critical joints* as defined in Section 4.3 without regard for the direction of the applied load.

Commentary:
See the Commentary to Section 3.3.1.

3.3.4. Long-Slotted Holes: When approved by the *Engineer of Record*, long-slotted holes are permitted in only one ply at any individual *faying surface* of *snug-tightened joints* as defined in Section 4.1, and *pretensioned joints* as defined in Section 4.2, provided the applied load is approximately perpendicular (between 80 and 100 degrees) to the axis of the slot. When approved by the *Engineer of Record*, long-slotted holes are permitted in one ply only at any individual *faying surface* of *slip-critical joints* as defined in Section 4.3 without regard for the direction of the applied load. Fully inserted finger shims between the *faying surfaces* of load-transmitting elements of bolted *joints* are not considered a long-slotted element of a *joint*; nor are they considered to be a ply at any individual *faying surface*.

Commentary:
See the Commentary to Section 3.3.1.

Finger shims are devices that are often used to permit the alignment and plumbing of structures. When these devices are fully and properly inserted, they do not have the same effect on bolt pretension relaxation or the *connection* performance, as do long-slotted holes in an outer ply. When fully inserted, the shim provides support around approximately 75 percent of the perimeter of the bolt in contrast to the greatly reduced area that exists with a bolt that is centered in a long slot. Furthermore, finger shims are always enclosed on both sides by the connected material, which should be effective in bridging the space between the fingers.

3.4. Burrs

Burrs that extend $^{1}/_{16}$ in. or less above the surface are permitted to remain on the *faying surfaces* of *snug-tightened joints* as defined in Section 4.1 and *pretensioned joints* as defined in Section 4.2. Burrs that extend over $^{1}/_{16}$ in. above the surface shall be removed from all *joints*. Burrs that would prevent solid seating of the connected plies prior to the pretensioning of *slip-critical joints* as defined in Section 4.3 shall be removed.

Commentary:

Polyzois and Yura (1985) and McKinney and Zwerneman (1993) demonstrated that the slip resistance of *joints* was either unchanged or slightly improved by the presence of burrs. Therefore, small ($^{1}/_{16}$ in. or less) burrs that do not prevent solid seating of the connected parts need not be removed. On the other hand, parallel tests in the same program demonstrated that large burrs (over $^{1}/_{16}$ in.) could cause a small increase in the required nut rotation from the snug-tight condition to achieve the specified pretension with the turn-of—nut pretensioning method. In the interest of simplicity, this Specification requires that all large burrs be removed.

SECTION 4. JOINT TYPE

For *joints* with fasteners that are loaded in shear or combined shear and tension, the *Engineer of Record* shall specify the *joint* type in the contract documents as snug-tightened, pretensioned or slip-critical. For *slip-critical joints*, the required class of slip resistance in accordance with Section 5.4 shall also be specified. For *joints* with fasteners that are loaded in tension only, the *Engineer of Record* shall specify the *joint* type in the contract documents as snug-tightened or pretensioned. Table 4.1 summarizes the applications and requirements of the three *joint* types.

Table 4.1. Summary of Applications and Requirements for Bolted Joints

Load Transfer	Application	Joint Type[a,b]	Faying Surface Prep.?	Install per Section	Inspect per Section	Arbitrate per Section 10?
Shear only	Resistance to shear load by shear/bearing	ST	No	8.1	9.1	No
	Resistance to shear by shear/bearing. Bolt pretension is required, but for reasons other than slip resistance.	PT	No	8.2	9.2	No
	Shear-load resistance by friction on faying surfaces is required.	SC	Yes[d]	8.2	9.3	If required to resolve dispute
Combined shear and tension	Resistance to shear load by shear/bearing. Tension load is static only.[c]	ST	No	8.1	9.1	No
	Resistance to shear by shear/bearing. Bolt pretension is required, but for reasons other than slip resistance.	PT	No	8.2	9.2	If required to resolve dispute
	Shear-load resistance by friction on faying surfaces is required.	SC	Yes[d]	8.2	9.3	If required to resolve dispute
Tension only	Static loading only.[c]	ST	No	8.1	9.1	No
	All other conditions of tension-only loading.	PT	No	8.2	9.2	If required to resolve dispute

[a] Under Joint Type: ST = snug-tightened, PT = pretensioned and SC = slip-critical; See Section 4.
[b] See Sections 4 and 5 for the design requirements for each joint type.
[c] Per Section 4.2, the use of ASTM A490 bolts in snug-tightened joints with tensile loads is not permitted.
[d] See Section 3.2.2.

Commentary:

When first approved by the Research Council on Structural Connections, in January, 195l, the "Specification for Assembly of Structural Joints Using High-Strength Bolts" merely permitted the substitution of a like number of ASTM A325 bolts for hot driven ASTM A141[2] steel rivets of the same nominal diameter. Additionally, it was required that all bolts be pretensioned and that all *faying surfaces* be free of paint; hence, satisfying the requirements for a *slip-critical joint* by the present-day definition. As revised in 1954, the omission of paint was required to apply only to "*joints* subject to stress reversal, impact or vibration, or to cases where stress redistribution due to *joint* slippage would be undesirable." This relaxation of the earlier provision recognized the fact that, in many applications, movement of the connected parts that brings the bolts into bearing against the sides of their holes is in no way detrimental. Bolted *joints* were then designated as "bearing type", "friction type" or "direct tension". With the 1985 edition of this Specification, these designations were changed to "shear/bearing", "slip-critical" and "direct tension", respectively, and snug-tightened installation was permitted for many *shear/bearing joints*. With this edition of this Specification, *snug-tightened joints* are also permitted for qualified applications involving ASTM A325 bolts in direct tension.

If non-pretensioned bolts are used in the type of *joint* that places the bolts in shear, load is transferred by shear in the bolts and bearing stress in the connected material. At the ultimate limit state, failure will occur by shear failure of the bolts, by bearing failure of the connected material or by failure of the member itself. On the other hand, if pretensioned bolts are used in such a *joint*, the frictional force that develops between the connected plies will initially transfer the load. Until the frictional force is exceeded, there is no shear in the bolts and no bearing stress in the connected components. Further increase of load places the bolts into shear and against the connected material in bearing, just as was the case when non-pretensioned bolts were used. Since it is known that the pretension in bolts will have been dissipated by the time bolt shear failure takes place (Kulak et al., 1987; p. 49), the ultimate limit state of a pretensioned bolted *joint* is the same as an otherwise identical *joint* that uses non-pretensioned bolts.

Because the consequences of slip into bearing vary from application to application, the determination of whether a *joint* can be designated as snug-tightened or as pre-tensioned or rather must be designated as slip-critical is best left to judgment and a decision on the part of the *Engineer of Record*. In the case of *joints* with three or more bolts in holes with only a small clearance, the freedom to slip generally does not exist. It is probable that normal fabrication tolerances and erection procedures are such that one or more bolts are in bearing even before additional load is applied. Such is the case for standard holes and for slotted holes loaded transverse to the axis of the slot.

Joints that are required to be *slip-critical joints* include:

(1) Those cases where slip movement could theoretically exceed an amount deemed by the *Engineer of Record* to affect the serviceability of the structure or through excessive distortion to cause a reduction in strength or stability, even though the resistance to fracture of the *connection* and yielding of the member may be adequate; and,

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[2] ASTM A141 (discontinued in 1967) became identified as A502 Grade 1 (discontinued 1999).

(2) Those cases where slip of any magnitude must be prevented, such as in *joints* subject to significant load reversal and *joints* between elements of built-up compression members in which any slip could cause a reduction of the flexural stiffness required for the stability of the built-up member.

In this Specification, the provisions for the design, installation and inspection of bolted *joints* are dependent upon the type of *joint* that is specified by the *Engineer of Record*. Consequently, it is required that the *Engineer of Record* identify the *joint* type in the contract documents.

4.1. Snug-Tightened Joints

Except as required in Sections 4.2 and 4.3, *snug-tightened joints* are permitted.

Bolts in *snug-tightened joints* shall be designed in accordance with the applicable provisions of Sections 5.1, 5.2 and 5.3, installed in accordance with Section 8.1 and inspected in accordance with Section 9.1. As indicated in Section 4 and Table 4.1, requirements for *faying surface* condition shall not apply to *snug-tightened joints*.

Commentary:

Recognizing that the ultimate strength of a *connection* is independent of the bolt pretension and slip movement, there are numerous practical cases in the design of structures where, if slip occurs, it will not be detrimental to the serviceability of the structure. Additionally, there are cases where slip of the *joint* is desirable to permit rotation in a *joint* or to minimize the transfer of moment. To provide for these cases while at the same time making use of the shear strength of *high-strength bolts*, *snug-tightened joints* are permitted.

The maximum amount of slip that can occur in a *joint* is, theoretically, equal to twice the hole clearance. In practical terms, it is observed in laboratory and field experience to be much less; usually, about one-half the hole clearance. Acceptable inaccuracies in the location of holes within a pattern of bolts usually cause one or more bolts to be in bearing in the initial, unloaded condition. Furthermore, even with perfectly positioned holes, the usual method of erection causes the weight of the connected elements to put some of the bolts into direct bearing at the time the member is supported on loose bolts and the lifting crane is unhooked. Additional loading in the same direction would not cause additional *joint* slip of any significance.

With this edition of this Specification, *snug-tightened joints* are also permitted for statically loaded applications involving ASTM A325 bolts and ASTM F1852 twist-off-type tension-control bolt assemblies in direct tension. However, snug-tightened installation is not permitted for these fasteners in applications involving non-static loading, nor for applications involving ASTM A490 bolts.

4.2. Pretensioned Joints

Pretensioned joints are only required in the following applications:

(1) *Joints* in which fastener pretension is required in the specification or code that invokes this Specification;
(2) *Joints* that are subject to significant load reversal;
(3) *Joints* that are subject to fatigue load with no reversal of the loading direction;
(4) *Joints* with ASTM A325 or F1852 bolts that are subject to tensile fatigue; and,
(5) *Joints* with ASTM A490 bolts that are subject to tension or combined shear and tension, with or without fatigue.

Bolts in *pretensioned joints* subject to shear shall be designed in accordance with the applicable provisions of Sections 5.1 and 5.3, installed in accordance with Section 8.2 and inspected in accordance with Section 9.2. Bolts in *pretensioned joints* subject to tension or combined shear and tension shall be designed in accordance with the applicable provisions of Sections 5.1, 5.2, 5.3 and 5.5, installed in accordance with Section 8.2 and inspected in accordance with Section 9.2. As indicated in Section 4 and Table 4.1, requirements for *faying surface* condition shall not apply to *pretensioned joints*.

Commentary:
Under the provisions of some other specifications, certain shear *connections* are required to be pretensioned, but are not required to be slip-critical. Several cases are given, for example, in AISC LRFD Specification Section J1.11 (AISC, 1999) wherein certain bolted *joints* in bearing *connections* are to be pretensioned regardless of whether or not the potential for slip is a concern. The AISC Specification requires that *joints* be pretensioned in the following circumstances:

(1) Column splices in buildings with high ratios of height to width;
(2) *Connections* of members that provide bracing to columns in tall buildings;
(3) Various *connections* in buildings with cranes over 5-ton capacity; and,
(4) *Connections* for supports of running machinery and other sources of impact or stress reversal.

When pretension is desired for reasons other than the necessity to prevent slip, a *pretensioned joint* should be specified in the contract documents.

4.3. Slip-Critical Joints

Slip-critical joints are only required in the following applications involving shear or combined shear and tension:

(1) *Joints* that are subject to fatigue load with reversal of the loading direction;
(2) *Joints* that utilize oversized holes;
(3) *Joints* that utilize slotted holes, except those with applied load approximately normal (within 80 to 100 degrees) to the direction of the long dimension of the slot; and,
(4) *Joints* in which slip at the *faying surfaces* would be detrimental to the performance of the structure.

Bolts in *slip-critical joints* shall be designed in accordance with the applicable provisions of Sections 5.1, 5.2, 5.3, 5.4 and 5.5, installed in accordance with Section 8.2 and inspected in accordance with Section 9.3.

Commentary:
In certain cases, slip of a bolted *joint* in shear under service loads would be undesirable or must be precluded. Clearly, *joints* that are subject to reversed fatigue load must be slip-critical since slip may result in back-and-forth movement of the *joint* and the potential for accelerated fatigue failure. Unless slip is intended, as desired in a sliding expansion *joint*, slip in *joints* with long-slotted holes that are parallel to the direction of the applied load might be large enough to invalidate structural analyses that are based upon the assumption of small displacements.

For *joints* subject to fatigue load with respect to shear of the bolts that does not involve a reversal of load direction, there are two alternatives for fatigue design. The designer can provide either a *slip-critical joint* that is proportioned on the basis of the applied stress range on the gross section, or a *pretensioned joint* that is proportioned on the basis of applied stress range on the net section.

SECTION 5. LIMIT STATES IN BOLTED JOINTS

The design shear strength and design tensile strength of bolts shall be determined in accordance with Section 5.1. The interaction of combined shear and tension on bolts shall be limited in accordance with Section 5.2. The design bearing strength of the connected parts at bolt holes shall be determined in accordance with Section 5.3. Each of these *design strengths* shall be equal to or greater than the *required strength*. The axial load in bolts that are subject to tension or combined shear and tension shall be calculated with consideration of the effects of the externally applied tensile load and any additional tension resulting from *prying action* produced by deformation of the connected parts.

When slip resistance is required at the *faying surfaces* subject to shear or combined shear and tension, slip resistance shall be checked at either the factored-load level or service-load level, at the option of the *Engineer of Record*. When slip of the *joint* under factored loads would affect the ability of the structure to support the factored loads, the *design strength* determined in accordance with Section 5.4.1 shall be equal to or greater than the *required strength*. When slip resistance under service loads is the design criterion, the strength determined in accordance with Section 5.4.2 shall be equal to or greater than the effect of the service loads. In addition, slip-critical connections must meet the strength requirements to resist the factored loads as shear/bearing joints. Therefore, the strength requirements of Sections 5.1, 5.2 and 5.3 shall also be met.

When bolts are subject to cyclic application of axial tension, the stress determined in accordance with Section 5.5 shall be equal to or greater than the stress due to the effect of the service loads, including any additional tension resulting from prying action produced by deformation of the connected parts.

Commentary:
This section of the Specification provides the design requirements for *high-strength bolts* in bolted *joints*. However, this information is not intended to provide comprehensive coverage of the design of *high-strength bolted connections*. Other design considerations of importance to the satisfactory performance of the connected material, such as block shear rupture, shear lag, *prying action* and *connection* stiffness and its effect on the performance of the structure, are beyond the scope of this Specification and Commentary.

The design of bolted *joints* that transmit shear requires consideration of the shear strength of the bolts and the bearing strength of the connected material. If such *joints* are designated as *slip-critical joints*, the slip resistance must also be checked. This serviceability check can be made at the factored-load level (Section 5.4.1) or at the service-load level (Section 5.4.2). Regardless of which load level is selected for the check of slip resistance, the prevention of slip in the service-load range is the design criterion.

Parameters that influence the shear strength of bolted *joints* include:

(1) Geometric parameters – the ratio of the net area to the gross area of the connected parts, the ratio of the net area of the connected parts to the total shear-resisting area of the bolts and the length of the *joint*; and,

(2) Material parameter – the ratio of the yield strength to the tensile strength of the connected parts.

Using both mathematical models and physical testing, it was possible to study the influences of these parameters (Kulak et al., 1987; pp. 89-116 and 126-132). These showed that, under the rules that existed at that time the longest (and often the most important) *joints* had the lowest factor of safety, about 2.0 based on ultimate strength.

In general, bolted *joints* that are designed in accordance with the provisions of this Specification will have a higher reliability than will the members they connect. This occurs primarily because the resistance factors used in limit states for the design of bolted *joints* were chosen to provide a reliability higher than that used for member design. Additionally, the controlling strength limit state in the structural member, such as yielding or deflection, is usually reached well before the strength limit state in the *connection*, such as bolt shear strength or bearing strength of the connected material. The installation requirements vary with *joint* type and influence the behavior of the *joints* within the service-load range, however, this influence is ignored in all strength calculations. Secondary tensile stresses that may be produced in bolts in *shear/bearing joints*, such as through the flexing of double-angle *connections* to accommodate the simple-beam end rotation, need not be considered.

It is sometimes necessary to use *high-strength bolts* and fillet welds in the same *connection*, particularly as the result of remedial work. When these fastening elements act in the same shear plane, the combined strength is a function of whether the bolts are snug-tightened or pretensioned, the location of the bolts relative to the holes in which they are located and the orientation of the fillet welds. The fillet welds can be parallel or transverse to the direction of load. Recent work (Manuel and Kulak, 1999) can be used to calculate the *design strength* of such *joints*.

5.1. Design Shear and Tensile Strengths

Shear and tensile strengths shall not be reduced by the installed bolt pretension. For *joints*, the design shear and tensile strengths shall be taken as the sum of the strengths of the individual bolts.

The design strength in shear or the design strength in tension for an ASTM A325, A490 or F1852 bolt is ϕR_n, where $\phi = 0.75$ and:

$$R_n = F_n A_b \qquad \text{(Equation 5.1)}$$

where

R_n = nominal strength (shear strength per shear plane or tensile strength) of a bolt, kips;

F_n = nominal strength per unit area from Table 5.1 for the appropriate applied load conditions, ksi, adjusted for the presence of fillers as fillers as required below, and,

A_b = cross-sectional area based upon the nominal diameter of bolt, in.^{2}

When a bolt that carries load passes through fillers or shims in a shear plane that are equal to or less than 1/4-in. thick, F_n from Table 5.1 shall be used without reduction. When a bolt that carries load passes through fillers or shims that are greater than 1/4-in. thick, they shall be designed in accordance with one of the following procedures:

Table 5.1. Nominal Strength per Unit Area of Bolts

Applied Load Condition		Nominal Strength per Unit Area F_n, ksi	
		ASTM A325 or F1852 Bolt	ASTM A490 Bolt
Tension[a]	Static	90	113
	Fatigue	See Section 5.5	
Shear[a,b]	Threads included in shear plane	48	60
	Threads excluded from shear plane	60	75

[a] Except as required in Section 5.2.
[b] In shear *connections* that transmit axial force and have length between extreme bolts measured parallel to the line of force exceeds 50 in., tabulated values shall be reduced by 20 percent.

(1) For fillers or shims that are equal to or less than ¾ in. thick, F_n from Table 5.1 shall be multiplied by the factor [1 - 0.4(t' - 0.25)], where t' is the total thickness of fillers or shims, in., up to ¾ in.;

(2) The fillers or shims shall be extended beyond the *joint* and the filler or shim extension shall be secured with enough bolts to uniformly distribute the total force in the connected element over the combined cross-section of the connected element and the fillers or shims;

(3) The size of the *joint* shall be increased to accommodate a number of bolts that is equivalent to the total number required in (2) above; or

(4) The *joint* shall be designed as a *slip-critical joint*. The slip resistance of the *joint* shall not be reduced for the presence of fillers or shims.

Commentary:

The nominal shear and tensile strengths of ASTM A325, F1852 and A490 bolts are given in Table 5.1. These values are based upon the work of a large number of researchers throughout the world, as reported in the *Guide* (Kulak et al., 1987). The *design strength* equals the *nominal strength* multiplied by a resistance factor ϕ. On average, the *design strengths* result in bolted *joint* designs that are approximately equivalent to those provided under the allowable stress rules given in the 1980 edition of this Specification.

The nominal shear strength is based upon the observation that the shear strength of a single *high-strength bolt* is about 0.62 times the tensile strength of that bolt (Kulak et al., 1987; pp. 44-50). However, in lap splices transmitting axial force between members with more than two bolts in the line of force, non-uniform deformation of the connected material between fasteners causes a non-uniform distribution of the shear force to the bolts. Consequently, the strength of the *joint* decreases in terms of the average strength of all the bolts in the *joint* as the *joint* length increases (Kulak et al., 1987; pp. 99-104). Rather than provide a decreasing function that reflects this decrease in average fastener strength with *joint* length, a single reduction factor of 0.80 is applied to the 0.62 multiplier. This accommodates bolts in all *joints* up to 50 in. in length without seriously affecting the economy of very short *joints*. As noted in Footnote b in Table 5.1, the average shear strength of bolts in *joints* longer than 50 in. in length must be further reduced by 20 percent. Note that this reduction does not apply in cases when the distribution of force is essentially uniform along the *joint*, such as the bolted *joints* in a shear *connection* at the end of a deep plate girder.

The average ratio of nominal shear strength for bolts with threads included in the shear plane to the nominal shear strength for bolts with threads excluded from the shear plane is 0.83 with a standard deviation of 0.03 (Frank and Yura, 1981). Conservatively, a reduction factor of 0.80 is used to account for the reduction in shear strength for a bolt with threads included in the shear plane but calculated with the area corresponding to the nominal bolt diameter. The case of a bolt in double shear with a non-threaded section in one shear plane and a threaded section in the other shear plane is not covered in this Specification for two reasons. First, the manner in which load is shared between these two dissimilar shear areas is uncertain. Second, the detailer's lack of certainty as to the orientation of the bolt placement might leave both shear planes in the threaded section. Thus, if threads are included in one shear plane, the conservative assumption is made that threads are included in all shear planes.

The tensile strength of a *high-strength bolt* is the product of its ultimate tensile strength (per unit area) and some area through the threaded portion. This area, called the tensile stress area, is a derived quantity that is a function of the relative thread size and pitch. For the usual sizes of structural bolts, it is about 75 percent of the nominal cross-sectional area of the bolt. Hence, the nominal tensile strengths per unit area given in Table 5.1 are 0.75 times the tensile strength of the bolt material. According to Equation 5.1, the nominal area of the bolt is then used to calculate the *design strength* in tension. The *nominal strengths* so-calculated are intended to form the basis for comparison with the externally applied bolt tension plus any additional tension that results from *prying action* that is produced by deformation of the connected elements.

If pretensioned bolts are used in a *joint* that loads the bolts in tension, the question arises as to whether the pretension and the applied tension are additive. Because the compressed parts are being unloaded during the application of the external tensile force, the increase in bolt tension is minimal until the parts separate (Kulak et al., 1987; pp. 263-266). Thus, there will be little increase in bolt force above the pretension load under service loads. After the parts separate, the bolt acts as a tension member, as expected, and its *design strength* is that given in Equation 5.1 multiplied by the resistance factor ϕ.

Pretensioned bolts have torsion present during the installation process. Once the installation is completed, any residual torsion is quite small and will disappear entirely when the fastener is loaded to the point of plate separation. Hence, there is no question of torsion—tension interaction when considering the ultimate tensile strength of a *high-strength bolt* (Kulak et al., 1987; pp. 41-47).

When required, pretension is induced in a bolt by imposing a small axial elongation during installation, as described in the Commentary to Section 8. When the *joint* is subsequently loaded in shear, tension or combined shear and tension, the bolts will undergo significant deformations prior to failure that have the effect of overriding the small axial elongation that was introduced during installation and, thereby, removing the pretension. Measurements taken in laboratory tests confirm that the pretension that would be sustained if the applied load were removed is essentially zero before the bolt fails in shear (Kulak et al., 1987; pp. 93-94). Thus, the shear and tensile strengths of a bolt are not affected by the presence of an initial pretension in the bolt.

See also the Commentary to Section 5.5.

5.2. Combined Shear and Tension

When combined shear and tension loads are transmitted by an ASTM A325, A490 or F1852 bolt, the ultimate limit-state interaction shall be:

$$\left[\frac{T_u}{(\phi R_n)_t}\right]^2 + \left[\frac{V_u}{(\phi R_n)_v}\right]^2 \leq 1 \qquad \text{(Equation 5.2)}$$

where

T_u = *required strength* in tension (factored tensile load) per bolt, kips;

V_u = *required strength* in shear (factored shear load) per bolt, kips;

$(\phi R_n)_t$ = *design strength* in tension determined in accordance with Section 5.1, kips; and,

$(\phi R_n)_v$ = *design strength* in shear determined in accordance with Section 5.1, kips

Commentary:

When both shear forces and tensile forces act on a bolt, the interaction can be conveniently expressed as an elliptical solution (Chesson et al., 1965) that includes the elements of the bolt acting in shear alone and the bolt acting in tension alone. Although the elliptical solution provides the best estimate of the strength of bolts subject to combined shear and tension and is thus used in this Specification, the nature of the elliptical solution is such that it can be approximated conveniently using three straight lines (Carter et al., 1997). Earlier editions of this specification have used such linear representations for the convenience of design calculations. The elliptical interaction equation in effect shows that, for design purposes, significant interaction does not occur until either force component exceeds 20 percent of the limiting strength for that component.

5.3. Design Bearing Strength at Bolt Holes

For *joints*, the design bearing strength shall be taken as the sum of the strengths of the connected material at the individual bolt holes.

The design bearing strength of the connected material at a standard bolt hole, oversized bolt hole, short-slotted bolt hole independent of the direction of loading or long-slotted bolt hole with the slot parallel to the direction of the bearing load is ϕR_n, where $\phi = 0.75$ and:

(1) when deformation of the bolt hole at service load is a design consideration;

$$R_n = 1.2 L_c t F_u \leq 2.4 d_b t F_u \qquad \text{(Equation 5.3)}$$

(2) when deformation of the bolt hole at service load is not a design consideration;

$$R_n = 1.5 L_c t F_u \le 3 d_b t F_u \qquad \text{(Equation 5.4)}$$

The design bearing strength of the connected material at a long-slotted bolt hole with the slot perpendicular to the direction of the bearing load is ϕR_n, where ϕ = 0.75 and:

$$R_n = L_c t F_u \le 2 d_b t F_u \qquad \text{(Equation 5.5)}$$

In Equations 5.3, 5.4 and 5.5,

R_n = *nominal strength* (bearing strength of the connected material), kips;
F_u = specified minimum tensile strength (per unit area) of the connected material, ksi;
L_c = clear distance, in the direction of load, between the edge of the hole and the edge of the adjacent hole or the edge of the material, in.;
d_b = nominal diameter of bolt, in.; and,
t = thickness of the connected material, in.

Commentary:
The contact pressure at the interface between a bolt and the connected material can be expressed as a bearing stress on the bolt or on the connected material. The connected material is always critical. For simplicity, the bearing area is expressed as the bolt diameter times the thickness of the connected material in bearing. The governing value of the bearing stress has been determined from extensive experimental research and a further limitation on strength was derived from the case of a bolt at the end of a tension member or near another fastener.

The design equations are based upon the models presented in the *Guide* (Kulak et al., 1987; pp. 141-143), except that the clear distance to another hole or edge is used in the Specification formulation rather than the bolt spacing or end distance as used in the *Guide* (see Figure C-5.1). Equation 5.3 is derived from tests (Kulak et al., 1987; pp. 112-116) that showed that the total elongation, including local bearing deformation, of a standard hole that is loaded to obtain the ultimate strength equal to $3d_b tF_u$ in Equation 5.4 was on the order of the diameter of the bolt. This apparent hole elongation results largely from bearing deformation of the material that is immediately adjacent to the bolt. The lower value of $2.4d_b tF_u$ in Equation 5.3 provides a bearing strength limit-state that is attainable at reasonable deformation (¼ in.). Strength and deformation limits were thus used to jointly evaluate bearing strength test results for design.

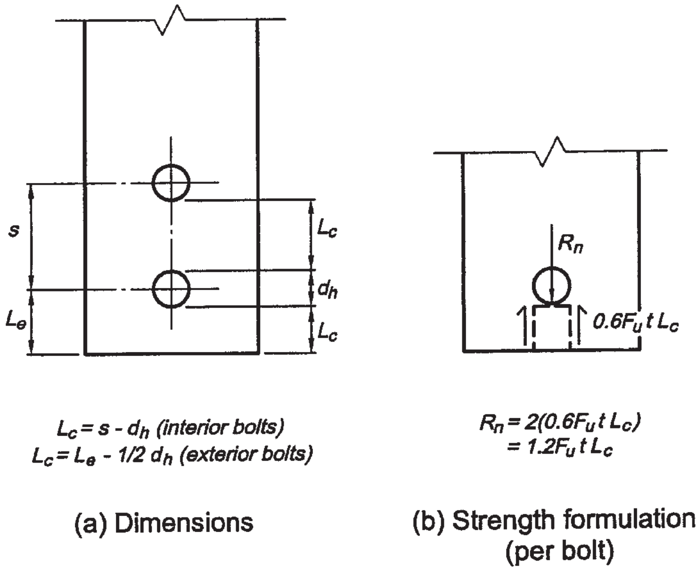


Figure. C-5.1. Bearing strength formulation.

When long-slotted holes are oriented with the long dimension perpendicular to the direction of load, the bending component of the deformation in the material between adjacent holes or between the hole and the edge of the plate is increased. The nominal bearing strength is limited to $2d_b t F_u$, which again provides a bearing strength limit-state that is attainable at reasonable deformation.

The design bearing strength has been expressed as that of a single bolt, although it is really that of the connected material that is immediately adjacent to the bolt. In calculating the design bearing strength of a connected part, the total bearing strength of the connected part can be taken as the sum of the bearing strengths of the individual bolts.

5.4. Design Slip Resistance

5.4.1. At the Factored-Load Level: The design slip resistance is ϕR_n, where ϕ is as defined below and:

$$R_n = \mu D_u T_m N_b \left(1 - \frac{T_u}{D_u T_m N_b}\right) \qquad \text{(Equation 5.6)}$$

where

ϕ = 1.0 for standard holes
= 0.85 for oversized and short-slotted holes
= 0.70 for long-slotted holes perpendicular to the direction of load
= 0.60 for long-slotted holes parallel to the direction of load;

R_n = *nominal strength* (slip resistance) of a slip plane, kips;

μ = *mean slip coefficient* for Class A, B or C *faying surfaces*, as applicable, or as established by testing in accordance with Appendix A (see Section 3.2.2(b))

= 0.33 for Class A *faying surfaces* (uncoated clean mill scale steel surfaces or surfaces with Class A coatings on blast-cleaned steel)

= 0.50 for Class B surfaces (uncoated blast-cleaned steel surfaces or surfaces with Class B coatings on blast-cleaned steel

= 0.35 for Class C surfaces (roughened hot-dip galvanized surfaces);

D_u = 1.13, a multiplier that reflects the ratio of the mean installed bolt pretension to the specified minimum bolt pretension T_m; the use of other values of D_u shall be approved by the *Engineer of Record;*

T_m = specified minimum bolt pretension (for *pretensioned joints* as specified in Table 8.1), kips;

N_b = number of bolts in the *joint*; and

T_u = *required strength* in tension (tensile component of applied factored load for combined shear and tension loading), kips

= zero if the *joint* is subject to shear only

5.4.2. At the Service-Load Level: The service-load slip resistance is ϕR_s, where ϕ is as defined in Section 5.4.1 and:

$$R_s = \mu D T_m N_b \left(1 - \frac{T}{D T_m N_b}\right) \qquad \text{(Equation 5.7)}$$

where

D = 0.80, a slip probability factor that reflects the distribution of actual slip coefficient values about the mean, the ratio of mean installed bolt pretension to the specified minimum bolt pretension, T_m, and a slip probability level; the use of other values of D must be approved by the *Engineer of Record*

T = applied service load in tension (tensile component of applied service load for combined shear and tension loading), kips

= zero if the *joint* is subject to shear only

and all other variables are as defined for Equation 5.6.

Commentary:
The design check for slip resistance can be made either at the factored-load level (Section 5.4.1) or at the service-load level (Section 5.4.2). These alternatives are based upon different design philosophies, which are discussed below. They have been calibrated to produce results that are essentially the same. The factored-load level approach is provided for the expedience of only working with factored loads. Irrespective of the approach, the limit state is based upon the prevention of slip at service-load levels.

If the factored-load provision is used, the *nominal strength* R_n represents the mean resistance, which is a function of the *mean slip coefficient* μ and the specified minimum bolt pretension (clamping force) T_m. The 1.13 multiplier in Equation 5.6 accounts for the expected 13 percent higher mean value of the installed bolt pretension provided by the calibrated wrench pretensioning method compared to the specified minimum bolt pretension T_m used in the calculation. In the absence of other field test data, this value is used for all methods.

If the service-load approach is used, a probability of slip is identified. It implies that there is 90 percent reliability that slip will not occur at the calculated slip load if the calibrated wrench pretensioning method is used, or that there is 95 percent reliability that slip will not occur at the calculated slip load if the turn-of-nut pretensioning method is used. The probability of loading occurrence was not considered in developing these slip probabilities (Kulak et al., 1987; pg. 135).

For most applications, the assumption that the slip resistance at each fastener is equal and additive with that at the other fasteners is based on the fact that all locations must develop the slip force before a total *joint* slip can occur at that plane. Similarly the forces developed at various slip planes do not necessarily develop simultaneously, but one can assume that the full slip resistances must be mobilized at each plane before full *joint* slip can occur. Equations 5.6 and 5.7 are formulated for the general case of a single slip plane. The total slip resistance of a *joint* with multiple slip planes can be calculated as that for a single slip plane multiplied by the number of slip planes.

Only the *Engineer of Record* can determine whether the potential slippage of a *joint* is critical at the service-load level as a serviceability consideration only or whether slippage could result in distortions of the frame such that the ability of the frame to resist the factored loads would be reduced. The following comments reflect the collective thinking of the Council and are provided as guidance and an indication of the intent of the Specification. See also the Commentary to Sections 4.2 and 4.3.

(1) If *joints* with standard holes have only one or two bolts in the direction of the applied load, a small slip may occur. In this case, *joints* subject to vibration should be proportioned to resist slip at the service-load level.

(2) In built-up compression members, such as double-angle struts in trusses, a small relative slip between the elements especially at the end *connections* can increase the effective length of the combined cross-section to that of the individual components and significantly reduce the compressive strength of the strut. Therefore, the *connection* between the elements at the ends of built-up members should be checked at the factored-load level, whether or not a *slip-critical joint* is required for serviceability. As given by Sherman and Yura (1998), the required slip resistance is $0.008P_uLQ/I$, where P_u is the axial compressive force in the built-up member, kips, L is the total length of the built-up member, in., Q is the first moment of area of one component about the axis of buckling of the built-up member, in.^{3}, and I is the moment of inertia of the built-up member about the axis of buckling, in.^{4}

(3) In *joints* with long-slotted holes that are parallel to the direction of the applied load, the designer has two alternatives. The *joint* can be designed to prevent slip in the service-load range using either the factored-load-level provision in Section 5.4.1 or the service-load-level provision in Section 5.4.2. In either case, however, the effect of the factored loads acting on the deformed structure (deformed by the maximum amount of slip in the long slots at all locations) must be included in the structural analysis.

(4) In *joints* subject to fatigue, design should be based upon service-load criteria and the design slip resistance of Section 5.4.2 because fatigue is a function of the service load performance rather than that of the factored load.

Extensive data developed through research sponsored by the Council and others during the past twenty years has been statistically analyzed to provide improved information on slip probability of *joints* in which the bolts have been pretensioned to the requirements of Table 8.1. Two variables, the *mean slip coefficient* of the *faying surfaces* and the bolt pretension, were found to affect the slip resistance of *joints*. Field studies (Kulak and Birkemoe, 1993) of installed bolts in various structural applications indicate that the Table 8.1 pretensions have been achieved as anticipated in the laboratory research.

An examination of the slip-coefficient data for a wide range of surface conditions indicates that the data are distributed normally and the standard deviation is essentially the same for each surface condition class. This means that different reduction factors should be applied to classes of surfaces with different *mean slip coefficients*—the smaller the mean value of the coefficient of friction, the smaller (more severe) the appropriate reduction factor—to provide equivalent reliability of slip resistance.

The bolt clamping force data indicate that bolt pretensions are distributed normally for each pretensioning method. However, the data also indicate that the mean value of the bolt pretension is different for each method. As noted previously, if the calibrated wrench method is used to pretension ASTM A325 bolts, the mean value of bolt pretension is about 1.13 times the specified minimum pretension in Table 8.1. If the turn-of-nut pretensioning method is used, the mean pretension is about 1.35 times the specified minimum pretension for ASTM A325 bolts and about 1.26 for ASTM A490 bolts.

The combined effects of the variability of the *mean slip coefficient* and bolt pretension have been accounted for approximately in the single value of the slip probability factor D in the equation for nominal slip resistance in Section 5.4.2. This implies 90 percent reliability that slip will not occur if the calibrated wrench pretensioning method is used and 95 percent reliability if the turn-of-nut pretensioning method is used. For values of D that are appropriate for other *mean slip coefficients* and slip probabilities, refer to the *Guide* (Kulak et al., 1987; pg. 135). The values given therein are suitable for direct substitution into the formula for slip resistance in Section 5.4.2.

The calibrated wrench installation method targets a specific bolt pretension, which is 5 percent greater than the specified minimum value given in Table 8.1. Thus, regardless of the actual strength of production bolts, this target value is unique for a given fastener grade. On the other hand, the turn-of-nut installation method imposes an elongation on the fastener. Consequently, the inherent strength of the bolts being installed will be reflected in the resulting pretension because this elongation will bring the fastener to its proportional limit under combined torsion and tension. As a result of these differences, the mean value and nature of the frequency distribution of pretensions for the two installation methods differ. Turn-of-nut installations result in higher mean levels of pretension than do calibrated wrench installations. These differences were taken into account when the design criteria for *slip-critical joints* were developed.

Statistical information on the pretension characteristics of bolts installed in the field using direct tension indicators and twist-off-type tension-control bolts is limited.

In any of the foregoing installation methods, it can be expected that a portion of the bolt assembly (the threaded portion of the bolt within the *grip* length and/or the engaged threads of the nut and bolt) will reach the inelastic region of behavior. This permanent distortion has no undesirable effect on the subsequent performance of the bolt.

Because of the greater likelihood that significant deformation can occur in *joints* with oversized or slotted holes, lower values of design slip resistance are provided for *joints* with these hole types through a modification of the resistance factor ϕ. For the case of long-slotted holes, even though the slip load is the same for loading transverse or parallel to the axis of the slot, the value for loading parallel to the axis has been further reduced, based upon judgment, in recognition of the greater consequences of slip.

Although the design philosophy for *slip-critical joints* presumes that they do not slip into bearing when subject to loads in the service range, it is mandatory that *slip-critical joints* also meet the requirements of Sections 5.1, 5.2 and 5.3. Thus, they must meet the strength requirements to resist the factored loads as *shear/bearing joints*.

Section 3.2.2(b) permits the *Engineer of Record* to authorize the use of *faying surfaces* with a *mean slip coefficient* μ that is less than 0.50 (Class B) and other than 0.33 (Class A) This authorization requires that the following restrictions are met:

(1) The *mean slip coefficient* μ must be determined in accordance with Appendix A; and,
(2) The appropriate slip probability factor *D* must be selected from the *Guide* (Kulak et al., 1987) for design at the service-load level.

Prior to the 1994 edition of this Specification, μ for Class C surfaces was taken as 0.40. This value was reduced to 0.35 in the 1994 edition for better agreement with the available research (Kulak et al., 1987; pp. 78-82).

5.5. Tensile Fatigue

The tensile stress in the bolt that results from the cyclic application of externally applied service loads and the prying force, if any, but not the pretension, shall not exceed the stress in Table 5.2. The nominal diameter of the bolt shall be used in

Table 5.2. Maximum Tensile Stress for Fatigue Loading

Number of Cycles	Maximum Bolt Stress for Design at Service Loads[a], ksi	
	ASTM A325 or F1852 Bolt	ASTM A490 Bolt
Not more than 20,000	44	54
From 20,000 to 500,000	40	49
More than 500,000	31	38

[a] Including the effects of *prying action*, if any, but excluding the pretension.

calculating the bolt stress. The connected parts shall be proportioned so that the calculated prying force does not exceed 30 percent of the externally applied load. *Joints* that are subject to tensile fatigue loading shall be specified as pretensioned in accordance with Section 4.2 or slip-critical in accordance with Section 4.3.

Commentary:
As described in the Commentary to Section 5.1, *high-strength bolts* in *pretensioned joints* that are nominally loaded in tension will experience little, if any, increase in axial stress under service loads. For this reason, pretensioned bolts are not adversely affected by repeated application of service-load tensile stress. However, care must be taken to ensure that the calculated prying force is a relatively small part of the total applied bolt tension (Kulak et al., 1987; p. 272). The provisions that cover bolt fatigue in tension are based upon research results where various single-bolt assemblies and *joints* with bolts in tension were subjected to repeated external loads that produced fatigue failure of the pretensioned fasteners. A limited range of prying effects was investigated in this research. As a matter of judgment, in this edition of the Specification the limit on prying forces as a percentage of the total externally applied tensile force has been reduced from 60 percent to 30 percent.

SECTION 6. USE OF WASHERS

6.1. Snug-Tightened Joints

Washers are not required in snug-tightened joints, except as required in Sections 6.1.1 and 6.1.2.

6.1.1. Sloping Surfaces: When the outer face of the *joint* has a slope that is greater than 1:20 with respect to a plane that is normal to the bolt axis, an ASTM F436 beveled washer shall be used to compensate for the lack of parallelism.

6.1.2. Slotted Hole: When a slotted hole occurs in an outer ply, an ASTM F436 washer or 5/16 in. thick common plate washer shall be used to cover the hole.

6.2 Pretensioned Joints and Slip-Critical Joints

Washers are not required in *pretensioned joints* and *slip-critical joints*, except as required in Sections 6.1.1, 6.1.2, 6.2.1, 6.2.2, 6.2.3, 6.2.4 and 6.2.5.

6.2.1. Specified Minimum Yield Strength of Connected Material Less Than 40 ksi: When ASTM A490 bolts are pretensioned in connected material of specified minimum yield strength less than 40 ksi, ASTM F436 washers shall be used under both the bolt head and nut, except that a washer is not needed under the head of an A490-strength round head twist-off bolt that meets the minimum bearing circle diameter requirements of ASTM F1852.

6.2.2. Calibrated Wrench Pretensioning: When the calibrated wrench pretensioning method is used, an ASTM F436 washer shall be used under the turned element.

6.2.3. Twist-Off-Type Tension-Control Bolt Pretensioning: When the twist-off-type tension-control bolt pretensioning method is used, an ASTM F436 washer shall be used under the nut as part of the *fastener assembly*.

6.2.4. Direct-Tension-Indicator Pretensioning: When the direct-tension-indicator pretensioning method is used, an ASTM F436 washer shall be used as follows:

(1) When the nut is turned and the direct tension indicator is located under the bolt head, an ASTM F436 washer shall be used under the nut;

(2) When the nut is turned and the direct tension indicator is located under the nut, an ASTM F436 washer shall be used between the nut and the direct tension indicator;

(3) When the bolt head is turned and the direct tension indicator is located under the nut, an ASTM F436 washer shall be used under the bolt head; and,

(4) When the bolt head is turned and the direct tension indicator is located under the bolt head, an ASTM F436 washer shall be used between the bolt head and the direct tension indicator.

6.2.5. Oversized or Slotted Hole: When an oversized or slotted hole occurs in an outer ply, the washer requirements shall be as given in Table 6.1. The washer used shall be of sufficient size to completely cover the hole.

Table 6.1. Washer Requirements for Bolted Joints with Oversized and Slotted Holes in the Outer Ply

<table>
<tr><th rowspan="2">ASTM Designation</th><th rowspan="2">Nominal Bolt Diameter, d_b, in.</th><th colspan="3">Hole Type in Outer Ply</th></tr>
<tr><th>Oversized</th><th>Short-Slotted</th><th>Long-Slotted</th></tr>
<tr><td>A325 or F1852</td><td>½-1½</td><td colspan="2" rowspan="2">ASTM F436[a]</td><td rowspan="2">5/16-in.-thick plate washer or continuous bar[b, c]</td></tr>
<tr><td rowspan="2">A490</td><td>≤ 1</td></tr>
<tr><td>> 1</td><td colspan="2">ASTM F436 with 5/16 in. thickness[b, d]</td><td>ASTM F436 washer with either a 3/8-in.-thick structural grade plate washer or continuous bar[b]</td></tr>
<tr><td colspan="5">[a] This requirement shall not apply to heads of round head tension-control bolt assemblies that meet the requirements in Section 2.7 and provide a bearing circle diameter that meets the requirements of ASTM F1852.
[b] Multiple washers with a combined thickness of 5/16 in. or larger do not satisfy this requirement.
[c] The plate washer or bar shall be of structural-grade steel material, but need not be hardened.
[d] Alternatively, a 3/8-in.-thick plate washer and an ordinary thickness F436 washer may be used. The plate washer need not be hardened.</td></tr>
</table>

Commentary:
It is important that shop drawings and *connection* details clearly reflect the number and disposition of washers when they are required, especially the thick hardened washers or plate washers that are required for some slotted hole applications. The total thickness of washers in the *grip* affects the length of bolt that must be supplied and used.

The primary function of washers is to provide a hardened non-galling surface under the turned element, particularly for torque-based pretensioning methods such as the calibrated wrench pretensioning method and twist-off-type tension-control bolt pretensioning method. Circular flat washers that meet the requirements of ASTM F436 provide both a hardened non-galling surface and an increase in bearing area that is approximately 50 percent larger than that provided by a heavy-hex bolt head or nut. However, tests have shown that washers of the standard 5/32 in. thickness have a minor influence on the pressure distribution of the induced bolt pretension. Furthermore, they showed that a larger thickness is required when ASTM A490 bolts are used with material that has a minimum specified yield strength that is less than 40 ksi. This is necessary to mitigate the effects of local yielding of the material in the vicinity of the contact area of the head and nut. The requirement for standard thickness hardened washers, when such washers are specified, is waived for alternative design fasteners that incorporate a bearing surface under the head of the same diameter as the hardened washer.

Heat-treated washers not less than 5/16 in. thick are required to cover over-sized and short-slotted holes in external plies, when ASTM A490 bolts of diameter larger than 1 in. are used, except per footnote d. This was found necessary to distribute the high clamping pressure so as to prevent collapse of the hole perimeter and enable the devel-opment of the desired clamping force. Preliminary investigation has shown that a similar but less severe deformation occurs when oversized or slotted holes are in the interior plies. The reduction in clamping force may be offset by “keying,” which tends to increase the resistance to slip. These effects are accentuated in *joints* of thin plies. When long-slotted holes occur in an outer ply, 3/8 in. thick plate washers or continuous bars and one ASTM F436 washer are required in Table 6.1. This requirement can be satisfied with material of any structural grade. Alternatively, either of the following options can be used:

(1) The use of material with F_y greater than 40 ksi will eliminate the need to also provide ASTM F436 washers in accordance with the requirements in Section 6.2.1 for ASTM A490 bolts of any diameter.

(2) Material with F_y equal to or less than 40 ksi can be used with ASTM F436 washers in accordance with the requirements in Section 6.2.1

This specification previously required a washer under bolt heads with a bearing area smaller than that provided by an F436 washer. Tests indicate that the pretension achieved with a bolt having the minimum F1852 bearing circle diameter is the same as that of a bolt with the larger bearing circle diameter equal to the size of an F436 washer, provided that the hole size meets the RCSC Specification limitations. (Schnupp, 2003)

SECTION 7. PRE-INSTALLATION VERIFICATION

The requirements in this Section shall apply only as indicated in Sections 8.2 to verify that the *fastener assemblies* and pretensioned installation procedures perform as required prior to installation.

7.1. Tension Calibrator

A *tension calibrator* shall be used where bolts are to be installed in *pretensioned joints* and *slip-critical joints* to:

(1) Confirm the suitability of the complete *fastener assembly*, including lubrication, for pretensioned installation; and,
(2) Confirm the procedure and proper use by the bolting crew of the pretensioning method to be used.

The accuracy of the *tension calibrator* shall be confirmed through calibration at least annually.

Commentary:
A *tension calibrator* is a hydraulic device that indicates the pretension that is developed in a bolt that is installed in it. Such a device is an economical and valuable tool and it must be readily available whenever *high-strength bolts* are to be pretensioned. A bolt *tension calibrator* is essential for:

(1) The pre-installation verification of the suitability of the *fastener assembly*, including the lubrication that is applied by the *manufacturer* or specially applied, to develop the specified minimum pretension;
(2) Verifying the adequacy and proper use of the specified pretensioning method to be used;
(3) Determining the installation torque for the calibrated wrench pretensioning method; and,
(4) Determining an arbitration torque as specified in Section 10, if required to resolve dispute.

It is the only economically available tool for the described essential uses in the shop and field.

Hydraulic *tension calibrators* undergo a slight deformation during bolt pretensioning. Hence, when bolts are pretensioned according to Section 8.2.1, the nut rotation corresponding to a given pretension reading may be somewhat larger than it would be if the same bolt were pretensioned in a solid steel assembly. Stated differently, the reading of an hydraulic *tension calibrator* tends to underestimate the pretension that a given rotation of the turned element would induce in a bolt in a *pretensioned joint*.

7.2. Required Testing

A representative sample of not fewer than three complete *fastener assemblies* of each combination of diameter, length, grade and *lot* to be used in the work shall be checked at the site of installation in a *tension calibrator* to verify that the pretensioning method develops a pretension that is equal to or greater than 1.05 times that specified for installation and inspection in Table 8.1. Washers shall be used in the pre-installation verification assemblies as required in the work in accordance with the requirements in Section 6.2.

If the actual pretension developed in any of the *fastener assemblies* is less than 1.05 times that specified for installation and inspection in Table 8.1, the cause(s) shall be determined and resolved before the *fastener assemblies* are used in the work. Cleaning, lubrication and retesting of these *fastener assemblies*, except ASTM F1852 twist-off-type tension-control bolt assemblies, (see Section 2.2) are permitted, provided that all assemblies are treated in the same manner.

Commentary:
The fastener components listed in Section 1.3 are manufactured under separate ASTM specifications, each of which includes tolerances that are appropriate for the individual component covered. While these tolerances are intended to provide for a reasonable and workable fit between the components when used in an assembly, the cumulative effect of the individual tolerances permits a significant variation in the installation characteristics of the complete *fastener assembly*. It is the intent in this Specification that the responsibility rests with the *supplier* for proper performance of the *fastener assembly*, the components of which may have been produced by more than one *manufacturer*.

When pretensioned installation is required, it is essential that the effects of the accumulation of tolerances, surface condition and lubrication be taken into account. Hence, pre-installation verification testing of the complete *fastener assembly* is required as indicated in Section 8 to ensure that the *fastener assemblies* and installation method to be used in the work will provide a pretension that exceeds those specified in Table 8.1. It is not, however, intended simply to verify conformance with the individual ASTM specifications.

It is recognized in this Specification that a natural scatter is found in the results of the pre-installation verification testing that is required in Section 8. Furthermore, it is recognized that the pretensions developed in tests of a representative sample of the fastener components that will be installed in the work must be slightly higher to provide confidence that the majority of *fastener assemblies* will achieve the minimum required pretension as given in Table 8.1. Accordingly, the minimum pretension to be used in pre-installation verification is 1.05 times that required for installation and inspection.

Pre-installation verification testing of as-received bolts and nuts is also a requirement in this Specification because of instances of under-strength and counterfeit bolts and nuts. Pre-installation verification testing provides a practical means for ensuring that non-conforming *fastener assemblies* are not incorporated into the work. Experience on many projects has shown that bolts and/or nuts not meeting the requirements of the applicable ASTM Specification would have been identified prior to installation if they had been tested as an assembly in a *tension calibrator*. The expense of replacing bolts installed in the structure when the non-conforming bolts were discovered at a later date would have been avoided.

Additionally, pre-installation verification testing clarifies for the bolting crew and the *inspector* the proper implementation of the selected pretensioning method and the adequacy of the installation equipment. It will also identify potential sources of problems, such as the need for lubrication to prevent failure of bolts by combined high torque with tension, under-strength assemblies resulting from excessive over-tapping of hot-dip galvanized nuts or other failures to meet strength or geometry requirements of applicable ASTM specifications.

The pre-installation verification requirements in this Section presume that *fastener assemblies* so verified will be pretensioned before the condition of the *fastener assemblies*, the equipment and the steelwork have changed significantly. Research by Kulak and Undershute (1998) on twist-off-type tension-control bolt assemblies from various *manufacturers* showed that installed pretensions could be a function of the time and environmental conditions of storage and exposure. The reduced performance of these bolts was caused by a deterioration of the lubricity of the assemblies. Furthermore, all bolt pre-tensioning that is achieved through rotation of the nut (or the head) is affected by the presence of torque, the excess of which has been demonstrated to adversely affect the development of the desired pretension. Thus, it is required that the condition of the *fastener assemblies* must be replicated in pre-installation verification. When time of exposure between the placement of *fastener assemblies* in the field work and the subsequent pretensioning of those *fastener assemblies* is of concern, pre-installation verification can be performed on *fastener assemblies* removed from the work or on extra *fastener assemblies* that, at the time of placement, were set aside to experience the same degree of exposure.

SECTION 8. INSTALLATION

Prior to installation, the fastener components shall be stored in accordance with Section 2.2. For *joints* that are designated in the contract documents as *snug-tightened joints*, the bolts shall be installed in accordance with Section 8.1. For *joints* that are designated in the contract documents as pretensioned or slip-critical, the bolts shall be installed in accordance with Section 8.2.

8.1. Snug-Tightened Joints

All bolt holes shall be aligned to permit insertion of the bolts without undue damage to the threads. Bolts shall be placed in all holes with washers positioned as required in Section 6.1 and nuts threaded to complete the assembly. Compacting the *joint* to the snug-tight condition shall progress systematically from the most rigid part of the *joint*. The snug-tightened condition is the tightness that is attained with a few impacts of an impact wrench or the full effort of an ironworker using an ordinary spud wrench to bring the connected plies into *firm contact*.

Commentary:

As discussed in the Commentary to Section 4, the bolted *joints* in most shear *connections* and in many tension *connections* can be specified as *snug-tightened joints*. The snug-tightened condition is the tightness that is attained with a few impacts of an impact wrench or the full effort of an ironworker using an ordinary spud wrench to bring the plies into *firm contact*. More than one cycle through the bolt pattern may be required to achieve the *snug-tightened joint*.

The actual pretensions that result in individual fasteners in *snug-tightened joints* will vary from *joint* to *joint* depending upon the thickness, flatness, and degree of parallelism of the connected plies as well as the effort applied. In most *joints*, plies of *joints* involving material of ordinary thickness and flatness can be drawn into complete contact at relatively low levels of pretension. However, in some *joints* in thick material, or in material with large burrs, it may not be possible to reach continuous contact throughout the *faying surface* area as is commonly achieved in *joints* of thinner plates. This is generally not detrimental to the performance of the *joint*.

As used in Section 8.1, the term “undue damage” is intended to mean damage that would be sufficient to render the product unfit for its intended use.

8.2. Pretensioned Joints

One of the pretensioning methods in Sections 8.2.1 through 8.2.4 shall be used, except when alternative-design fasteners that meet the requirements of Section 2.8 or alternative washer-type indicating devices that meet the requirements of Section 2.6.2 are used, in which case, installation instructions provided by the *manufacturer* and approved by the *Engineer of Record* shall be followed. When it is impractical to turn the nut, pretensioning by turning the bolt head is permitted while rotation of the nut is prevented, provided that the washer requirements in Section 6.2 are met. A pretension that is equal to or greater than the value in Table 8.1 shall be provided. The pre-installation verification procedures specified in Section 7 shall be performed using *fastener assemblies* that are representative of the condition of those that will be pretensioned in the work.

Table 8.1. Minimum Bolt Pretension for Pretensioned and Slip-Critical Joints

Nominal Bolt Diameter d_b, in.	Specified Minimum Bolt Pretension T_m, kips[a]	
	ASTM A325 and F1852 Bolts	ASTM A490 Bolts
1/2	12	15
5/8	19	24
3/4	28	35
7/8	39	49
1	51	64
1 1/8	56	80
1 1/4	71	102
1 3/8	85	121
1 1/2	103	148

[a] Equal to 70 percent of the specified minimum tensile strength of bolts as specified in ASTM Specifications for tests of full-size ASTM A325 and A490 bolts with UNC threads loaded in axial tension, rounded to the nearest kip.

Commentary:
The minimum pretension for ASTM A325 and A490 bolts is equal to 70 percent of the specified minimum tensile strength. As tabulated in Table 8.1, the values have been rounded to the nearest kip.

Four pretensioning methods are provided without preference in this Specification. Each method may be relied upon to provide satisfactory results when conscientiously implemented with the specified *fastener assembly* components in good condition. However, it must be recognized that misuse or abuse is possible with any method. With all methods, it is important to first install bolts in all holes of the *joint* and to compact the *joint* until the connected plies are in *firm contact*. Only after completion of this operation can the *joint* be reliably pretensioned. Both the initial phase of compacting the *joint* and the subsequent phase of pretensioning should begin at the most rigidly fixed or stiffest point.

In some *joints* in thick material, it may not be possible to reach continuous contact throughout the *faying surface* area, as is commonly achieved in *joints* of thinner plates. This is not detrimental to the performance of the *joint*. If the specified pretension is present in all bolts of the completed *joint*, the clamping force, which is equal to the total of the pretensions in all bolts, will be transferred at the locations that are in contact and the *joint* will be fully effective in resisting slip through friction.

If individual bolts are pretensioned in a single continuous operation in a *joint* that has not first been properly compacted or fitted up, the pretension in the bolts that are pretensioned first may be relaxed or removed by the pretensioning of adjacent bolts. The resulting reduction in total clamping force will reduce the slip resistance.

In the case of hot-dip galvanized coatings, especially if the *joint* consists of many plies of thickly coated material, relaxation of bolt pretension may be significant and re-pretensioning of the bolts may be required subsequent to the initial pretensioning. Munse (1967) showed that a loss of pretension of approximately 6.5 percent occurred for galvanized plates and bolts due to relaxation as compared with 2.5 percent for uncoated *joints*. This loss of bolt pretension occurred in five days; loss recorded thereafter was negligible. Either this loss can be allowed for in design or pretension may be brought back to the prescribed level by re-pretensioning the bolts after an initial period of "settling-in".

As stated in the *Guide*, Kulak et al (1987; p. 61), "…it seems reasonable to expect an increase in bolt force relaxation as the *grip* length is decreased. Similarly, increasing the number of plies for a constant *grip* length might also lead to an increase in bolt relaxation."

8.2.1. Turn-of-Nut Pretensioning: All bolts shall be installed in accordance with the requirements in Section 8.1, with washers positioned as required in Section 6.2. Subsequently, the nut or head rotation specified in Table 8.2 shall be applied to all *fastener assemblies* in the *joint*, progressing systematically from the most rigid part of the *joint* in a manner that will minimize relaxation of previously pretensioned bolts. The part not turned by the wrench shall be prevented from rotating during this operation.

Table 8.2. Nut Rotation from Snug-Tight Condition for Turn-of-Nut Pretensioning [a,b]

Bolt Length[c]	Disposition of Outer Face of Bolted Parts		
	Both faces normal to bolt axis	One face normal to bolt axis, other sloped not more than 1:20[d]	Both faces sloped not more than 1:20 from normal to bolt axis[d]
Not more than $4d_b$	⅓ turn	½ turn	⅔ turn
More than $4d_b$ but not more than $8d_b$	½ turn	⅔ turn	⅚ turn
More than $8d_b$ but not more than $12d_b$	⅔ turn	⅚ turn	1 turn

[a] Nut rotation is relative to bolt regardless of the element (nut or bolt) being turned. For required nut rotations of ½ turn and less, the tolerance is plus or minus 30 degrees; for required nut rotations of ⅔ turn and more, the tolerance is plus or minus 45 degrees.

[b] Applicable only to joints in which all material within the grip is steel.

[c] When the bolt length exceeds $12d_b$, the required nut rotation shall be determined by actual testing in a suitable tension calibrator that simulates the conditions of solidly fitting steel.

[d] Beveled washer not used.

Commentary:
The turn-of-nut pretensioning method results in more uniform bolt pretensions than is generally provided with torque-controlled pretensioning methods. Strain-control that reaches the inelastic region of bolt behavior is inherently more reliable than a method that is dependent upon torque control. However, proper implementation is dependent upon ensuring that the *joint* is properly compacted prior to application of the required partial turn and that the bolt head (or nut) is securely held when the nut (or bolt head) is being turned.

Match-marking of the nut and protruding end of the bolt after snug-tightening can be helpful in the subsequent installation process, and is certainly an aid to inspection.

As indicated in Table 8.2, there is no available research that establishes the required nut rotation for bolt lengths exceeding $12d_b$. The required turn for such bolts can be established on a case-by-case basis using a *tension calibrator*.

8.2.2. Calibrated Wrench Pretensioning: The pre-installation verification procedures specified in Section 7 shall be performed daily for the calibration of the installation wrench. Torque values determined from tables or from equations that claim to relate torque to pretension without verification shall not be used.

All bolts shall be installed in accordance with the requirements in Section 8.1, with washers positioned as required in Section 6.2. Subsequently, the installation torque determined in the pre-installation verification of the *fastener assembly* (Section 7) shall be applied to all bolts in the *joint*, progressing systematically from the most rigid part of the *joint* in a manner that will minimize relaxation of previously pretensioned bolts. The part not turned by the wrench shall be prevented from rotating during this operation. Application of the installation torque need not produce a relative rotation between the bolt and nut that is greater than the rotation specified in Table 8.2.

Commentary:
The scatter in installed pretension can be significant when torque-controlled methods of installation are used. The variables that affect the relationship between torque and pretension include:

(1) The finish and tolerance on the bolt and nut threads;
(2) The uniformity, degree and condition of lubrication;
(3) The shop or job-site conditions that contribute to dust and dirt or corrosion on the threads;
(4) The friction that exists to a varying degree between the turned element (the nut face or bearing area of the bolt head) and the supporting surface;
(5) The variability of the air supply parameters on impact wrenches that results from the length of air lines or number of wrenches operating from the same source;
(6) The condition, lubrication and power supply for the torque wrench, which may change within a work shift; and,
(7) The repeatability of the performance of any wrench that senses or responds to the level of the applied torque.

In the first edition of this Specification, which was published in 1951, a table of torque-to-pretension relationships for bolts of various diameters was included. It was soon demonstrated in research that a variation in the torque-to-pretension of as high as ±40 percent must be anticipated unless the relationship is established individually for each bolt *lot*, diameter, and fastener condition. Hence, in the 1954 edition of this Specification, recognition of relationships between torque and pretension in the form of tabulated values or equations was withdrawn. Recognition of the calibrated wrench pretensioning method was retained however until 1980, but with the requirement that the torque required for installation be determined specifically for the bolts being installed on a daily basis. Recognition of the method was withdrawn in 1980 because of the continuing controversy that resulted from the failure of users to adhere to the requirements for the valid use of the method during both installation and inspection.

In the 1985 edition of this Specification, the calibrated wrench pretensioning method was reinstated, but with more emphasis on detailed requirements that must be carefully followed. For calibrated wrench pretensioning, wrenches must be calibrated:

(1) Daily;
(2) When the *lot* of any component of the *fastener assembly* is changed;
(3) When the *lot* of any component of the *fastener assembly* is relubricated;
(4) When significant differences are noted in the surface condition of the bolt threads, nuts or washers; or,
(5) When any major component of the wrench including lubrication, hose and air supply are altered.

It is also important that:

(1) Fastener components be protected from dirt and moisture at the shop or job-site as required in Section 2;
(2) Washers be used as specified in Section 6; and
(3) The time between removal from *protected storage* and wrench calibration and final pretensioning be minimal.

8.2.3. Twist-Off-Type Tension-Control Bolt Pretensioning: Twist-off-type tension-control bolt assemblies that meet the requirements of ASTM F1852 shall be used.

All *fastener assemblies* shall be installed in accordance with the requirements in Section 8.1 without severing the splined end and with washers positioned as required in Section 6.2. If a splined end is severed during this operation, the *fastener assembly* shall be removed and replaced. Subsequently, all bolts in the *joint* shall be pretensioned with the twist-off-type tension-control bolt installation wrench, progressing systematically from the most rigid part of the *joint* in a manner that will minimize relaxation of previously pretensioned bolts.

Commentary:
ASTM F1852 twist-off-type tension-control bolt assemblies have a splined end that extends beyond the threaded portion of the bolt. During installation, this

splined end is gripped by a specially designed wrench chuck and provides a means for turning the nut relative to the bolt. This product is, in fact, based upon a torque-controlled installation method to which the *fastener assembly* variables affecting torque that were discussed in the Commentary to Section 8.2.2 apply, except for wrench calibration, because torque is controlled within the *fastener assembly*.

Twist-off-type tension-control bolt assemblies must be used in the as-delivered, clean, lubricated condition as specified in Section 2. Adherence to the requirements in this Specification, especially those for storage, cleanliness and verification, is necessary for their proper use.

8.2.4. Direct-Tension-Indicator Pretensioning: Direct tension indicators that meet the requirements of ASTM F959 shall be used. The pre-installation verification procedures specified in Section 7 shall demonstrate that, when the pretension in the bolt reaches 1.05 times that specified for installation and inspection in Table 8.1, the gap is not less than the job inspection gap in accordance with ASTM F959.

All bolts shall be installed in accordance with the requirements in Section 8.1, with washers positioned as required in Section 6.2. The installer shall verify that the direct-tension-indicator protrusions have not been compressed to a gap that is less than the job inspection gap during this operation, and if this has occurred, the direct tension indicator shall be removed and replaced. Subsequently, all bolts in the *joint* shall be pretensioned, progressing systematically from the most rigid part of the *joint* in a manner that will minimize relaxation of previously pretensioned bolts. The installer shall verify that the direct tension indicator protrusions have been compressed to a gap that is less than the job inspection gap.

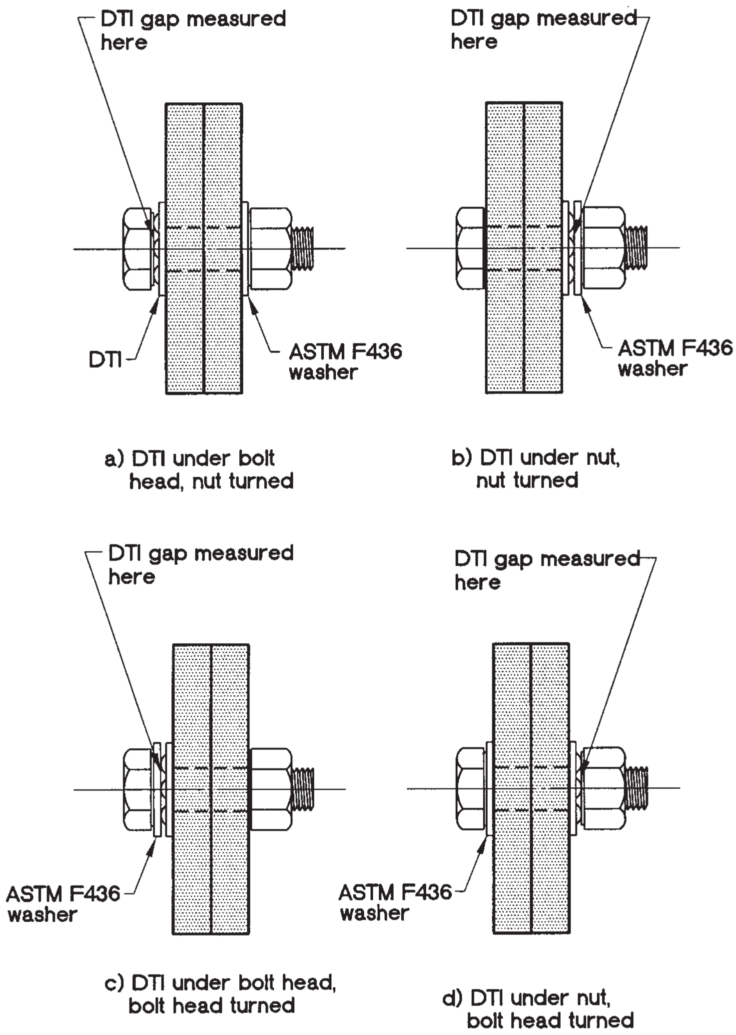
Commentary:

ASTM F959 direct tension indicators are recognized in this Specification as a bolt-tension-indicating device. Direct tension indicators are hardened, washer-shaped devices incorporating small arch-like protrusions on the bearing surface that are designed to deform in a controlled manner when subjected to compressive load.

During installation, care must be taken to ensure that the direct-tension-indicator arches are oriented to bear against the hardened bearing surface of the bolt head or nut or against a hardened flat washer if used under turned element whether that turned element is the nut or the bolt. Proper use and orientation is illustrated in Figure C-8.1.

In some cases, more than a single cycle of systematic partial pretensioning may be required to deform the direct-tension-indicator protrusions to the gap that is specified by the *manufacturer*. If the gaps fail to close or when the washer *lot* is changed, another verification procedure using the *tension calibrator* must be performed.

Provided the connected plies are in *firm contact*, partial compression of the direct tension indicator protrusions is commonly taken as an indication that the snug-tight condition has been achieved.



Note: See Section 6, for general requirements for the use of washers.

Figure. C-8.1. Proper use and orientation of ASTM F959 direct-tension indicator.

SECTION 9. INSPECTION

When inspection is required in the contract documents, the *inspector* shall ensure while the work is in progress that the requirements in this Specification are met. When inspection is not required in the contract documents, the *contractor* shall ensure while the work is in progress that the requirements in this Specification are met.

For *joints* that are designated in the contract documents as *snug-tightened joints*, the inspection shall be in accordance with Section 9.1. For *joints* that are designated in the contract documents as pretensioned, the inspection shall be in accordance with Section 9.2. For *joints* that are designated in the contract documents as slip-critical, the inspection shall be in accordance with Section 9.3.

9.1. Snug-Tightened Joints

Prior to the *start of work*, it shall be ensured that all fastener components to be used in the work meet the requirements in Section 2. Subsequently, it shall be ensured that all connected plies meet the requirements in Section 3.1 and all bolt holes meet the requirements in Sections 3.3 and 3.4. After the *connections* have been assembled, it shall be visually ensured that the plies of the connected elements have been brought into *firm contact* and that washers have been used as required in Section 6. No further evidence of conformity is required for *snug-tightened joints*. The magnitude of the clamping force that exists in a *snug-tightened joint* is not a consideration.

Commentary:
Inspection requirements for *snug-tightened joints* consist of verification that the proper fastener components were used, the connected elements were fabricated properly, and the bolted *joint* was drawn into firm contact. Because pretension is not required for the proper performance of a *snug-tightened joint*, the installed bolts should not be inspected to determine the actual installed pretension. Likewise, the arbitration procedures described in Section 10 are not appropriate.

9.2. Pretensioned Joints

For *pretensioned joints*, the following inspection shall be performed in addition to that required in Section 9.1:

(1) When the turn-of-nut pretensioning method is used for installation, the inspection shall be in accordance with Section 9.2.1.
(2) When the calibrated wrench pretensioning method is used for installation, the inspection shall be in accordance with Section 9.2.2.
(3) When the twist-off-type tension-control bolt pretensioning method is used for installation, the inspection shall be in accordance with Section 9.2.3.
(4) When the direct-tension-indicator pretensioning method is used for installation, the inspection shall be in accordance with Section 9.2.4.
(5) When alternative-design fasteners that meet the requirements of Section 2.8 or alternative washer-type indicating devices that meet the requirements of Section 2.6.2 are used, the inspection shall be in accordance with inspection instructions provided by the manufacturer and approved by the Engineer of Record.

Commentary:
When *joints* are designated as pretensioned, they are not subject to the same faying-surface-treatment inspection requirements as is specified for *slip-critical joints* in Section 9.3.

9.2.1. Turn-of-Nut Pretensioning: The *inspector* shall observe the pre-installation verification testing required in Section 8.2.1. Subsequently, it shall be ensured by *routine observation* that the bolting crew properly rotates the turned element relative to the unturned element by the amount specified in Table 8.2. Alternatively, when *fastener assemblies* are match-marked after the initial fit-up of the *joint* but prior to pretensioning, visual inspection after pretensioning is permitted in lieu of routine observation. No further evidence of conformity is required. A pretension that is greater than the value specified in Table 8.1 shall not be cause for rejection.

Commentary:
Match-marking of the assembly during installation as discussed in the Commentary to Section 8.2.1 improves the ability to inspect bolts that have been pretensioned with the turn-of-nut pretensioning method. The sides of nuts and bolt heads that have been impacted sufficiently to induce the Table 8.1 minimum pretension will appear slightly peened.

The turn-of-nut pretensioning method, when properly applied and verified during the construction, provides more reliable installed pretensions than after-the-fact *inspection* testing. Therefore, proper inspection of the method is for the inspector to observe the required pre-installation verification testing of the *fastener assemblies* and the method to be used, followed by monitoring of the work in progress to ensure that the method is routinely and properly applied, or visual inspection of match-marked assemblies.

Some problems with the turn-of-nut pretensioning method have been encountered with hot-dip galvanized bolts. In some cases, the problems have been attributed to an especially effective lubricant applied by the *manufacturer* to ensure that bolts and nuts from stock will meet the ASTM Specification requirements for minimum turns testing of galvanized fasteners. Job-site testing in the *tension calibrator* demonstrated that the lubricant reduced the coefficient of friction between the bolt and nut to the degree that “the full effort of an ironworker using an ordinary spud wrench” to snug-tighten the *joint* actually induced the full required pretension. Also, because the nuts could be removed with an ordinary spud wrench, they were erroneously judged by the *inspector* to be improperly pretensioned. Excessively lubricated *high-strength bolts* may require significantly less torque to induce the specified pretension. The required pre-installation verification will reveal this potential problem.

Conversely, the absence of lubrication or lack of proper over-tapping can cause seizing of the nut and bolt threads, which will result in a twist failure of the bolt at less than the specified pretension. For such situations, the use of a *tension calibrator* to check the bolt assemblies to be installed will be helpful in establishing the need for lubrication.

9.2.2. Calibrated Wrench Pretensioning: The *inspector* shall observe the pre-installation verification testing required in Section 8.2.2. Subsequently, it shall be ensured by *routine observation* that the bolting crew properly applies the calibrated wrench to the turned element. No further evidence of conformity is required. A pretension that is greater than the value specified in Table 8.1 shall not be cause for rejection.

Commentary:
For proper inspection of the method, it is necessary for the *inspector* to observe the required pre-installation verification testing of the *fastener assemblies* and the method to be used, followed by monitoring of the work in progress to ensure that the method is routinely and properly applied within the limits on time between removal from *protected storage* and final pretensioning.

9.2.3. Twist-Off-Type Tension-Control Bolt Pretensioning: The *inspector* shall observe the pre-installation verification testing required in Section 8.2.3. Subsequently, it shall be ensured by *routine observation* that the splined ends are properly severed during installation by the bolting crew. No further evidence of conformity is required. A pretension that is greater than the value specified in Table 8.1 shall not be cause for rejection.

Commentary:
The sheared-off splined end of an installed ASTM F1852 twist-off-type tension-control bolt assembly merely signifies that at some time the bolt was subjected to a torque that was adequate to cause the shearing. If in fact all fasteners are individually pretensioned in a single continuous operation without first properly snug-tightening all fasteners, they may give a misleading indication that the bolts have been properly pretensioned. Therefore it is necessary that the *inspector* observe the required pre-installation verification testing of the *fastener assemblies*, and the ability to apply partial tension prior to twist-off is demonstrated. This is followed by monitoring of the work in progress to ensure that the method is routinely and properly applied within the limits on time between removal from *protected storage* and final twist-off of the splined end.

9.2.4. Direct-Tension-Indicator Pretensioning: The *inspector* shall observe the pre-installation verification testing required in Section 8.2.4. Subsequently, but prior to pretensioning, it shall be ensured by *routine observation* that the appropriate feeler gage is accepted in at least half of the spaces between the protrusions of the direct tension indicator and that the protrusions are properly oriented away from the work. If the appropriate feeler gage is accepted in fewer than half of the spaces, the direct tension indicator shall be removed and replaced. After pretensioning, it shall be ensured by *routine observation* that the appropriate feeler gage is refused entry into at least half of the spaces between the protrusions. No further evidence of conformity is required. A pretension that is greater than that specified in Table 8.1 shall not be cause for rejection.

Commentary:
When the *joint* is initially snug tightened, the direct tension indicator arch-like protrusions will generally compress partially. Whenever the snug-tightening operation causes one-half or more of the gaps between these arch-like protrusions to close to 0.015 in. or less (0.005 in. or less for coated direct tension indicators), the direct tension indicator should be replaced. Only after this initial operation should the bolts be pretensioned in a systematic manner. If the bolts are installed and pretensioned in a single continuous operation, direct tension indicators may give the *inspector* a misleading indication that the bolts have been properly pretensioned. Therefore, it is necessary that the *inspector* observe the required pre-installation verification testing of the *fastener assemblies* with the direct-tension indicators properly located and the method to be used. Following this operation, the *inspector* should monitor the work in progress to ensure that the method is routinely and properly applied.

9.3. Slip-Critical Joints

Prior to assembly, it shall be visually verified that the *faying surfaces* of *slip-critical joints* meet the requirements in Section 3.2.2. Subsequently, the inspection required in Section 9.2 shall be performed.

Commentary:
When *joints* are specified as slip-critical, it is necessary to verify that the *faying surface* condition meets the requirements as specified in the contract documents prior to assembly of the *joint* and that the bolts are properly pretensioned after they have been installed. Accordingly, the inspection requirements for *slip-critical joints* are identical to those specified in Section 9.2, with additional *faying surface* condition inspection requirements.

SECTION 10. ARBITRATION

When it is suspected after inspection in accordance with Section 9.2 or Section 9.3 that bolts in pretensioned or *slip-critical joints* do not have the proper pretension, the following arbitration procedure is permitted. If verification of bolt pretension is required after the passage of a period of time and exposure of the completed *joints*, an alternative arbitration procedure that is appropriate to the specific situation shall be used.

(1) A representative sample of five bolt and nut assemblies of each combination of diameter, length, grade and *lot* in question shall be installed in a *tension calibrator*. The material under the turned element shall be the same as in the actual installation; that is, structural steel or hardened washer. The bolt shall be partially pretensioned to approximately 15 percent of the pretension specified in Table 8.1. Subsequently, the bolt shall be pretensioned to the minimum value specified in Table 8.1.

(2) A manual torque wrench that indicates torque by means of a dial, or one that may be adjusted to give an indication that a defined torque has been reached, shall be applied to the pretensioned bolt. The torque that is necessary to rotate the nut or bolt head five degrees (approximately 1 in. at 12-in. radius) relative to its mating component in the tightening direction shall be determined. The arbitration torque shall be determined by rejecting the high and low values and averaging the remaining three.

(3) Bolts represented by the above sample shall be tested by applying, in the tightening direction, the arbitration torque to 10 percent of the bolts, but no fewer than two bolts, selected at random in each *joint* in question. If no nut or bolt head is turned relative to its mating component by application of the arbitration torque, the *joint* shall be accepted as properly pretensioned.

If any nut or bolt is turned relative to its mating component by an attempted application of the arbitration torque, all bolts in the *joint* shall be tested. Those bolts whose nut or head is turned relative to its mating component by application of the arbitration torque shall be re-pretensioned by the Fabricator or Erector and reinspected. Alternatively, the Fabricator or Erector, at their option, is permitted to re-pretension all of the bolts in the *joint* and subsequently resubmit the *joint* for inspection.

Commentary:

When bolt pretension is arbitrated using torque wrenches after pretensioning, such arbitration is subject to all of the uncertainties of torque-controlled calibrated wrench installation that are discussed in the Commentary to Section 8.2.2. Additionally, the reliability of after-the-fact torque wrench arbitration is reduced by the absence of many of the controls that are necessary to minimize the variability of the torque-to-pretension relationship, such as:

(1) The use of hardened washers[3];

(2) Careful attention to lubrication; and,

(3) The uncertainty of the effect of passage of time and exposure in the installed condition.

Furthermore, in many cases such arbitration may have to be based upon an arbitration torque that is determined either using bolts that can only be assumed to be representative of the bolts used in the actual job or using bolts that are removed from completed *joints*. Ultimately, such arbitration may wrongly reject bolts that were subjected to a properly implemented installation procedure. The arbitration procedure contained in this Specification is provided, in spite of its limitations, as the most feasible available at this time.

Arbitration using an ultrasonic extensometer or a mechanical one capable of measuring changes in bolt length can be performed on a sample of bolts that is representative of those that have been installed in the work. Several *manufacturers* produce equipment specifically for this application. The use of appropriate techniques, which includes calibration, can produce a very accurate measurement of the actual pretension. The method involves measurement of the change in bolt length during the release of the nut, combined with either a load calibration of the removed *fastener assembly* or a theoretical calculation of the force corresponding to the measured elastic release or "stretch". Reinstallation of the released bolt or installation of a replacement bolt is required.

The required release suggests that the direct use of extensometers as an inspection tool be used in only the most critical cases. The problem of reinstallation may require bolt replacement unless torque can be applied slowly using a manual or hydraulic wrench, which will permit the restoration of the original elongation.

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[3] For example, because the reliability of the turn-of-nut pretensioning method is not dependent upon the presence or absence of washers under the turned element, washers are not generally required, except for other reasons as indicated in Section 6. Thus, in the absence of washers,after-the-fact, torque-based arbitration is particularly unreliable when the turn-of-nut pretensioning method has been used for installation.

APPENDIX A. TESTING METHOD TO DETERMINE THE SLIP COEFFICIENT FOR COATINGS USED IN BOLTED JOINTS

SECTION A1. GENERAL PROVISIONS

A1.1. Purpose and Scope

The purpose of this testing procedure is to determine the *mean slip coefficient* of a coating for use in the design of *slip-critical joints*. Adherence to this testing method provides that the creep deformation of the coating due to both the clamping force of the bolt and the service-load *joint* shear are such that the coating will provide satisfactory performance under sustained loading.

Commentary:

The Research Council on Structural Connections on June 14, 1984, first approved the testing method developed by Yura and Frank (1985). It has since been revised to incorporate changes resulting from the intervening years of experience with the testing method, and is now included as an appendix to this Specification.

The slip coefficient under short-term static loading has been found to be independent of the magnitude of the clamping force, variations in coating thickness and bolt hole diameter.

The proposed test methods are designed to provide the necessary information to evaluate the suitability of a coating for *slip-critical joints* and to determine the *mean slip coefficient* to be used in the design of the *joints*. The initial testing of the compression specimens provides a measure of the scatter of the slip coefficient.

The creep tests are designed to measure the creep behavior of the coating under the service loads, determined by the slip coefficient of the coating based upon the compression test results. The slip test conducted at the conclusion of the creep test is to ensure that the loss of clamping force in the bolt does not reduce the slip load below that associated with the design slip coefficient. ASTM A490 bolts are specified, since the loss of clamping force is larger for these bolts than that for ASTM A325 bolts. Qualification of the coating for use in a structure at an average thickness of 2 mils less than that to be used for the test specimen is to ensure that a casual buildup of the coating due to overspray and other causes does not jeopardize the coating's performance.

A1.2. Definition of Essential Variables

Essential variables are those that, if changed, will require retesting of the coating to determine its *mean slip coefficient*. The essential variables and the relationship of these variables to the limitations of application of the coating for structural *joints* are given below. The slip coefficient testing shall be repeated if there is any change in these essential variables.

A1.2.1. Time Interval: The time interval between application of the coating and the time of testing is an essential variable. The time interval must be recorded in hours and any special curing procedures detailed. Curing according to published *manufacturer's* recommendations would not be considered a special curing procedure. The coatings are qualified for use in structural *connections* that are assembled after coating for a time equal to or greater than the interval used in the test specimens. Special curing conditions used in the test specimens will also apply to the use of the coating in the structural *connections*.

A1.2.2. Coating Thickness: The coating thickness is an essential variable. The maximum average coating thickness, as per SSPC PA2 (SSPC 1993; SSPC 1991), allowed on the faying surfaces is 2 mils less than the average thickness, rounded to the nearest whole mil, of the coating that is used on the test specimens.

A1.2.3. Coating Composition and Method of Manufacture: The composition of the coating, including the thinners used, and its method of manufacture are essential variables.

A1.3. Retesting

A coating that fails to meet the creep or the post-creep slip test requirements in Section A4 may be retested in accordance with methods in Section A4 at a lower slip coefficient without repeating the static short-term tests specified in Section A3. Essential variables shall remain unchanged in the retest.

SECTION A2. TEST PLATES AND COATING OF THE SPECIMENS

A2.1. Test Plates

The test specimen plates for the short-term static tests are shown in Figure A1. The plates are 4 in. × 4 in. × ⅝ in. thick, with a 1 in. diameter hole drilled 1½ in. ± 1/16 in. from one edge. The test specimen plates for the creep tests are shown in Figure A2. The plates are 4 in. × 7 in. × ⅝ in. thick with two 1 in. diameter holes drilled 1½ in. ± 1/16 in. from each end. The edges of the plates may be milled, as-rolled or saw-cut; thermally cut edges are not permitted. The plates shall be flat enough to ensure that they will be in reasonably full contact over the *faying surface*. All burrs, lips or rough edges shall be removed. The arrangement of the specimen plates for the testing is shown in Figure A2. The plates shall be fabricated from a steel with a specified minimum yield strength that is between 36 and 50 ksi.

If specimens with more than one bolt are desired, the contact surface per bolt shall be 4 in. × 3 in. as shown for the single-bolt specimen in Figure A1.

Commentary:

The use of 1-in.-diameter bolt holes in the specimens is to ensure that adequate clearance is available for slip. Fabrication tolerances, coating buildup on the holes, and assembly tolerances tend to reduce the apparent clearances.

A2.2. Specimen Coating

Coatings are to be applied to the specimens in a manner that is consistent with that to be used in the actual intended structural application. The method of applying the coating and the surface preparation shall be given in the test report. The specimens are to be coated to an average thickness that is 2 mils greater than the maximum thickness to be used in the structure on both of the plate surfaces (the faying and outer surfaces). The thickness of the total coating and the primer, if used, shall be measured on the contact surface of the specimens. The thickness shall be measured in accordance with SSPC-PA2 (SSPC, 1993; SSPC, 1991). Two spot readings (six gage readings) shall be made for each contact surface. The overall average thickness from the three plates comprising a specimen is the average thickness for the specimen. This value shall be reported for each specimen. The average coating thickness of the creep specimens shall be calculated and reported.

The time between application of the coating and specimen assembly shall be the same for all specimens within ±4 hours. The average time shall be calculated and reported.

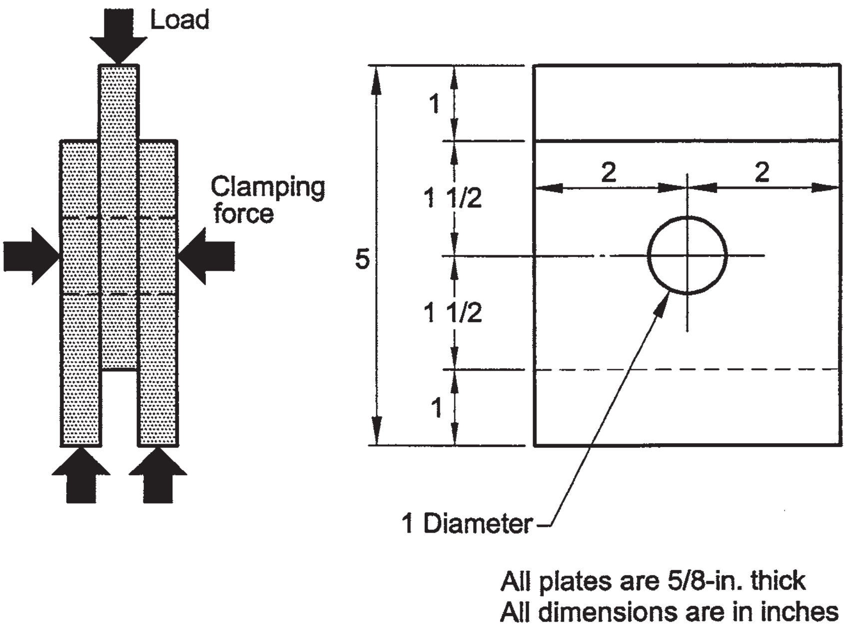

Figure A-1. Compression slip test specimen.

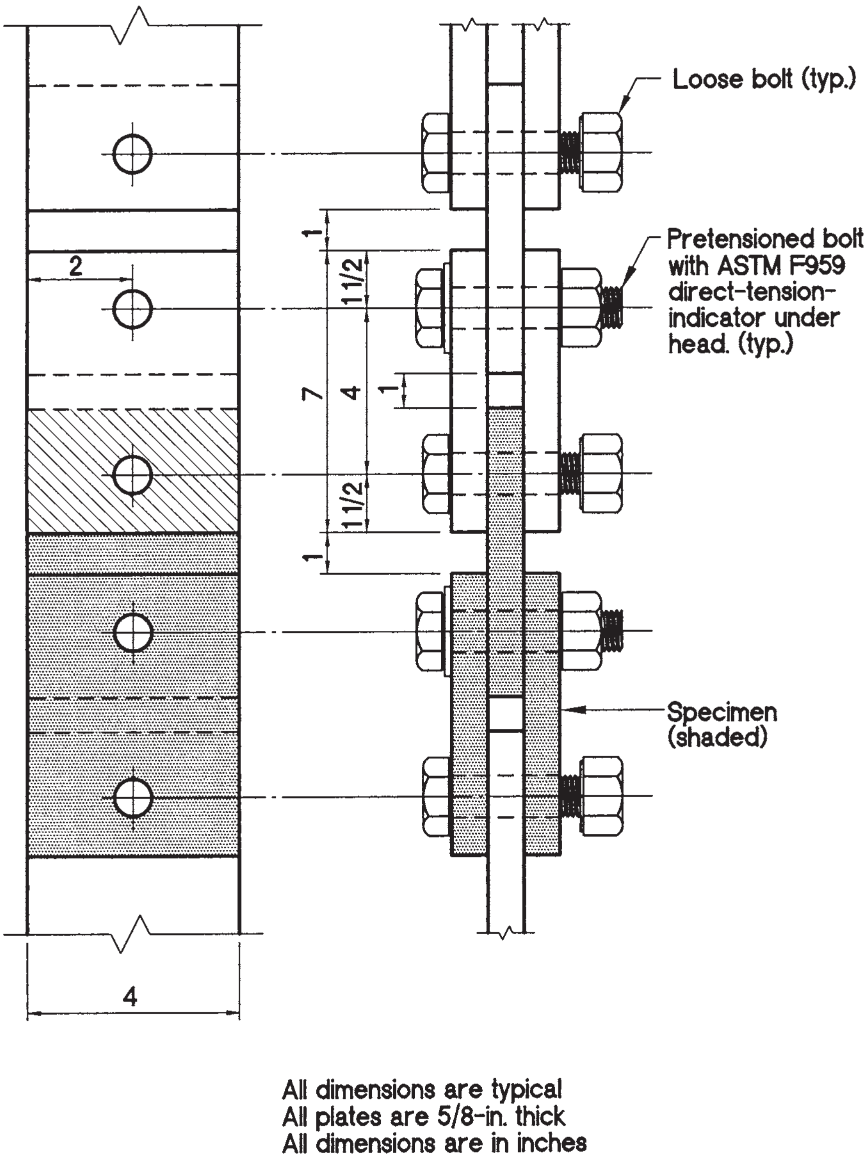

Figure A-2. Creep test specimen assembly.

SECTION A3. SLIP TESTS

The methods and procedures described herein are used to experimentally determine the *mean slip coefficient* under short-term static loading for *high-strength bolted joints*. The *mean slip coefficient* shall be determined by testing one set of five specimens.

Commentary:
The slip load measured in this setup yields the slip coefficient directly since the clamping force is controlled and measured directly. The resulting slip coefficient has been found to correlate with both tension and compression tests of bolted specimens. However, tests of bolted specimens revealed that the clamping force may not be constant but decreases with time due to the compressive creep of the coating on the *faying surfaces* and under the nut and bolt head. The reduction in clamping force can be considerable for *joints* with high clamping force and thick coatings (as much as a 20 percent loss). This reduction in clamping force causes a corresponding reduction in the slip load. The resulting reduction in slip load must be considered in the procedure used to determine the design allowable slip loads for the coating.

The loss in clamping force is a characteristic of the coating. Consequently, it cannot be accounted for by an increase in the factor of safety or a reduction in the clamping force used for design without unduly penalizing coatings that do not exhibit this behavior.

A3.1. Compression Test Setup

The test setup shown in Figure A3 has two major loading components, one to apply a clamping force to the specimen plates and another to apply a compressive load to the specimen so that the load is transferred across the *faying surfaces* by friction.

A3.1.1. Clamping Force System: The clamping force system consists of a ⅞ in. diameter threaded rod that passes through the specimen and a centerhole compression ram. An ASTM A563 grade DH nut is used at both ends of the rod and a hardened washer is used at each side of the test specimen. Between the ram and the specimen is a specially modified ⅞ in. diameter ASTM A563 grade DH nut in which the threads have been drilled out so that it will slide with little resistance along the rod. When oil is pumped into the centerhole ram, the piston rod extends, thus forcing the special nut against one of the outside plates of the specimen. This action puts tension in the threaded rod and applies a clamping force to the specimen, thereby simulating the effect of a pretensioned bolt. If the diameter of the centerhole ram is greater than 1 in., additional plate washers will be necessary at the ends of the ram. The clamping force system shall have a capability to apply a load of at least 49 kips and shall maintain this load during the test with an accuracy of 0.5 kips.

Commentary:
The slip coefficient can be easily determined using the hydraulic bolt test setup included in this Specification. The clamping force system simulates the clamping action of a pretensioned *high-strength bolt*. The centerhole ram applies a clamping force to the specimen, simulating that due to a pretensioned bolt.

A3.1.2. Compressive Load System: A compressive load shall be applied to the specimen until slip occurs. This compressive load shall be applied with a compression test machine or a reaction frame using a hydraulic loading device. The loading device and the necessary supporting elements shall be able to support a force of 120 kips. The compression loading system shall have a minimum accuracy of 1 percent of the slip load.

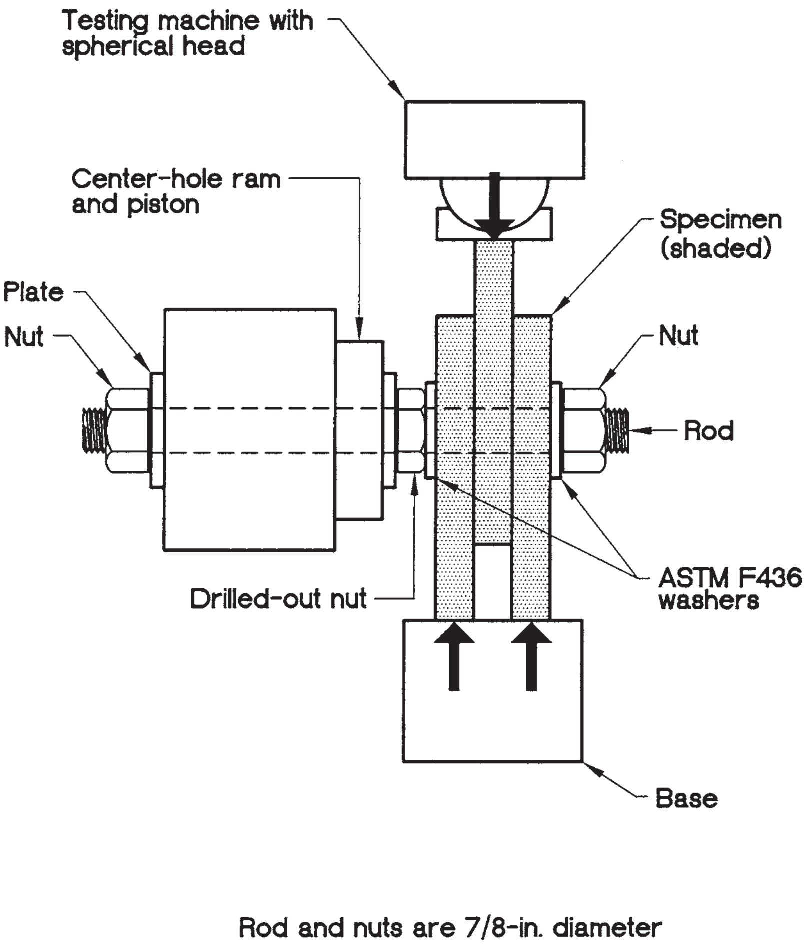

Figure A-3. Compression slip test setup.

A3.2. Instrumentation

A3.2.1. Clamping Force: The clamping force shall be measured within 0.5 kips. This is accomplished by measuring the pressure in the calibrated ram or placing a load cell in series with the ram.

A3.2.2. Compression Load: The compression load shall be measured during the test by direct reading from a compression testing machine, a load cell in series with the specimen and the compression loading device or pressure readings on a calibrated compression ram.

A3.2.3. Slip Deformation: The displacement of the center plate relative to the two outside plates shall be measured. This displacement, called "slip" for simplicity, shall be the average or that which occurs at the centerline of the specimen. This can be accomplished by using the average of two gages placed on the two exposed edges of the specimen or by monitoring the movement of the loading head relative to the base. If the latter method is used, due regard shall be taken for any slack that may be present in the loading system prior to application of the load. Deflections shall be measured by dial gages or any other calibrated device that has an accuracy of at least 0.001 in.

A3.3. Test Procedure

The specimen shall be installed in the test setup as shown in Figure A3. Before the hydraulic clamping force is applied, the individual plates shall be positioned so that they are in, or close to, full bearing contact with the ⅞ in. threaded rod in a direction that is opposite to the planned compressive loading to ensure obvious slip deformation. Care shall be taken in positioning the two outside plates so that the specimen is perpendicular to the base with both plates in contact with the base. After the plates are positioned, the centerhole ram shall be engaged to produce a clamping force of 49 kips. The applied clamping force shall be maintained within ±0.5 kips during the test until slip occurs.

The spherical head of the compression loading machine shall be brought into contact with the center plate of the specimen after the clamping force is applied. The spherical head or other appropriate device ensures concentric loading. When 1 kip or less of compressive load is applied, the slip gages shall be engaged or attached. The purpose of engaging the deflection gage(s), after a slight load is applied, is to eliminate initial specimen settling deformation from the slip reading.

When the slip gages are in place, the compression load shall be applied at a rate that does not exceed 25 kips per minute nor 0.003 in. of slip displacement per minute until the slip load is reached. The test should be terminated when a slip of 0.05 in. or greater is recorded. The load-slip relationship should preferably be monitored continuously on an X-Y plotter throughout the test, but in lieu of continuous data, sufficient load-slip data shall be recorded to evaluate the slip load defined below.

A3.4. Slip Load

Typical load-slip response is shown in Figure A4. Three types of curves are usually observed and the slip load associated with each type is defined as follows:

Curve (a)	Slip load is the maximum load, provided this maximum occurs before a slip of 0.02 in. is recorded.
Curve (b)	Slip load is the load at which the slip rate increases suddenly.
Curve (c)	Slip load is the load corresponding to a deformation of 0.02 in. This definition applies when the load vs. slip curves show a gradual change in response.

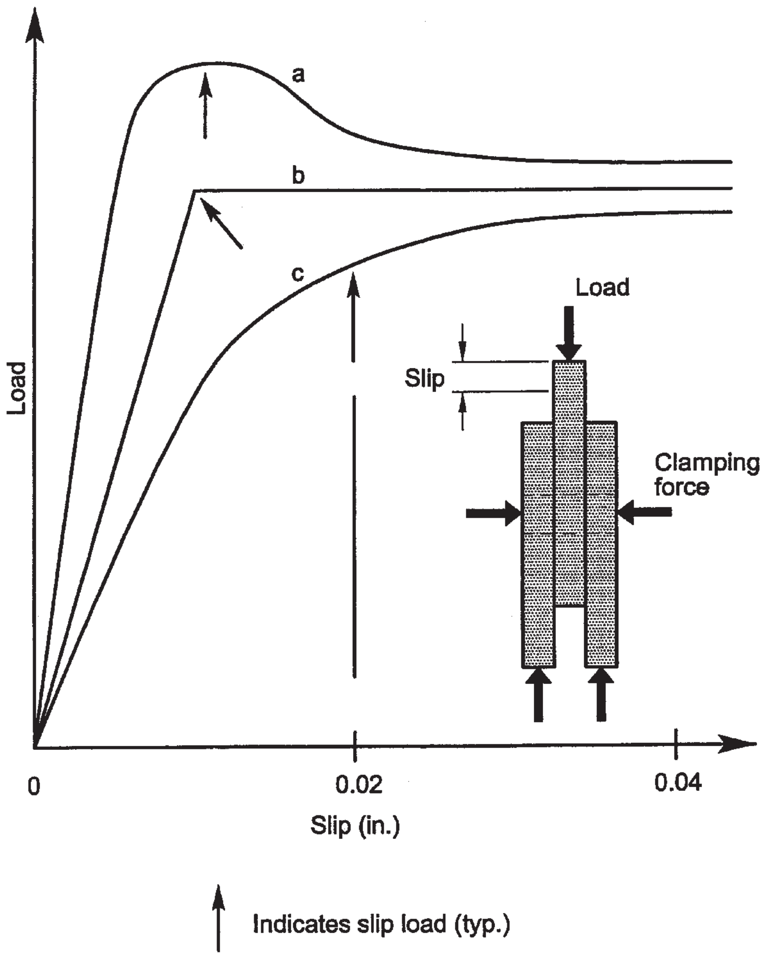

Figure A-4. Definition of slip load.

A3.5. Slip Coefficient

The slip coefficient for an individual specimen k_s shall be calculated as follows: The *mean slip coefficient* μ for one set of five specimens shall be reported.

$$k_s = \frac{slip\ load}{2 \times\ clamping\ force} \qquad \text{(Equation A3.1)}$$

A3.6. Alternative Test Methods

Alternative test methods to determine slip are permitted, provided the accuracy of load measurement and clamping satisfies the conditions presented in the previous sections. For example, the slip load may be determined from a tension-type test setup rather than the compression-type test setup as long as the contact surface area per bolt of the test specimen is the same as that shown in Figure A1. The clamping force of at least 49 kips may be applied by any means, provided the force can be established within ± 1 percent.

Commentary:

Alternative test procedures and specimens may be used as long as the accuracy of load measurement and specimen geometry are maintained as prescribed. For example, strain-gaged bolts can usually provide the desired accuracy. However, bolts that are pretensioned by the turn-of-nut, calibrated wrench, alternative-design fastener, or direct-tension-indicator pretensioning method usually show too much variation to meet the ± 1 percent requirement of the slip test.

SECTION A4. TENSION CREEP TEST

The test method outlined is intended to ensure that the coating will not undergo significant creep deformation under sustained service loading. The test also indicates the loss in clamping force in the bolt due to the compression or creep of the coating. Three replicate specimens are to be tested.

Commentary:
The creep deformation of the bolted *joint* under the applied shear loading is also an important characteristic and a function of the coating applied. Thicker coatings tend to creep more than thinner coatings. Rate of creep deformation increases as the applied load approaches the slip load. Extensive testing has shown that the rate of creep is not constant with time, rather it decreases with time. After about 1,000 hours of loading, the additional creep deformation is negligible.

A4.1. Test Setup

Tension-type specimens, as shown in Figure A2, are to be used. The replicate specimens are to be linked together in a single chain-like arrangement, using loose pin bolts, so the same load is applied to all specimens. The specimens shall be assembled so the specimen plates are bearing against the bolt in a direction opposite to the applied tension loading. Care shall be taken in the assembly of the specimens to ensure the centerline of the holes used to accept the pin bolts is in line with the bolts used to assemble the *joint*. The load level, specified in Section A4.2, shall be maintained constant within ±1 percent by springs, load maintainers, servo controllers, dead weight or other suitable equipment. The bolts used to clamp the specimens together shall be ⅞ in. diameter ASTM A490 bolts. All bolts shall come from the same *lot*.

The clamping force in the bolts shall be a minimum of 49 kips. The clamping force shall be determined by calibrating the bolt force with bolt elongation, if standard bolts are used. Alternatively, special *fastener assemblies* that control the clamping force by other means, such as calibrated bolt torque or strain gages, are permitted. A minimum of three bolt calibrations shall be performed using the technique selected for bolt force determination. The average of the three-bolt calibration shall be calculated and reported. The method of measuring bolt force shall ensure the clamping force is within ±2 kips of the average value.

The relative slip between the outside plates and the center plates shall be measured to an accuracy of 0.001 in. These slips are to be measured on both sides of each specimen.

A4.2. Test Procedure

The load to be placed on the creep specimens is the service load permitted for ⅞ in. diameter ASTM A490 bolts in *slip-critical joints* in Section 5 for the particular slip coefficient category under consideration. The load shall be placed on the specimen and held for 1,000 hours. The creep deformation of a specimen is calculated using the average reading of the two displacements on either side of the specimen. The difference between the average after 1,000 hours and the initial average reading taken within one-half hour after loading the specimens is defined as the creep deformation of the specimen. This value shall be reported for each specimen. If the creep deformation of any specimen exceeds 0.005 in., the coating has failed the test for the slip coefficient used. The coating may be retested using new specimens in accordance with this Section at a load corresponding to a lower value of slip coefficient.

If the value of creep deformation is less than 0.005 in. for all specimens, the specimens shall be loaded in tension to a load that is equal to the average clamping force times the design slip coefficient times 2, since there are two slip planes. The average slip deformation that occurs at this load shall be less than 0.015 in. for the three specimens. If the deformation is greater than this value, the coating is considered to have failed to meet the requirements for the particular *mean slip coefficient* used. The value of deformation for each specimen shall be reported.

Commentary:
See Commentary in Section A1.1

APPENDIX B. ALLOWABLE STRESS DESIGN (ASD) ALTERNATIVE

As an alternative to the load and resistance factor design provisions given in Sections 1 through 10, the following allowable stress design provisions are permitted. The provisions in Sections 1 through 10 in this Specification shall apply to ASD, except as follows:

B1.2 Loads, Load Factors and Load Combinations

The design and construction of the structure shall conform to an applicable allowable stress design specification for steel structures. When permitted in the applicable building code or specification, the allowable stresses in Section B5 are permitted to be increased to account for the effects of multiple transient loads in combination. When a load reduction factor is used to account for the effects of multiple transient loads in combination, the allowable stresses in Section B5 shall not be increased.

Commentary:
Although loads, load factors and load combinations are not explicitly specified in this Specification, the allowable stresses herein are based upon those specified in ASCE 7. When the design is governed by other load criteria, the allowable stresses specified herein shall be adjusted as appropriate.

SECTION B5. LIMIT STATES IN BOLTED JOINTS

The allowable shear strength and the allowable tensile strength of bolts shall be determined in accordance with Section B5.1. The interaction of combined shear and tension on bolts shall be limited in accordance with Section B5.2. The allowable bearing strength of the connected parts at bolt holes shall be determined in accordance with Section B5.3. Each of these allowable strengths shall be equal to or greater than the effect of the service loads. The axial load in bolts that are subject to tension or combined shear and tension shall be calculated with consideration of the externally applied tensile load and any additional tension resulting from *prying action* produced by deformation of the connected parts.

When slip resistance is required at the *faying surfaces* subject to shear or combined shear and tension, the slip resistance determined in accordance with Section B5.4 shall be equal to or greater than the effect of the service loads. In addition, the strength requirements in Sections B5.1, B5.2 and B5.3 shall also be met.

When bolts are subject to cyclic application of axial tension, the allowable stress determined in accordance with Section B5.5 shall be equal to or greater than the stress due to the effect of the service loads, including any additional tension resulting from *prying action* produced by deformation of the connected parts. In addition, the strength requirements in Sections B5.1, B5.2 and B5.3 shall also be met.

B5.1. Allowable Shear and Tensile Stresses

Shear and tensile strengths shall not be reduced by the installed bolt pretension. For *joints*, the allowable strength shall be based upon the allowable shear and tensile stresses of the individual bolts and shall be taken as the sum of the allowable strengths of the individual bolts.

The allowable shear strength or allowable tensile strength for an ASTM A325, A490 or F1852 bolt is R_a, where:

$$R_a = F_a A_b \qquad \text{(Equation B5.1)}$$

where

R_a	=	allowable shear strength per shear plane or allowable tensile strength of a bolt, kips;
F_a	=	allowable stress from Table B5.1 for the appropriate applied load conditions, ksi, adjusted for the presence of fillers or shims as required below; and,
A_b	=	cross-sectional area based upon the nominal diameter of bolt, in.^{2}

When a bolt that carries load passes through fillers or shims in a shear plane that are equal to or less than ¼ in. thick, F_a from Table B5.1 shall be used without reduction. When a bolt that carries load passes through fillers or shims that are greater than ¼ in. thick, one of the following requirements shall apply:

(1) For fillers or shims that are equal to or less than ¾ in. thick, F_a from Table B5.1 shall be multiplied by the factor $[1 - 0.4(t' - 0.25)]$, where t' is the total thickness of fillers or shims, in., up to ¾ in.;

(2) The fillers or shims shall be extended beyond the *joint* and the filler extension shall be secured with enough bolts to uniformly distribute the total force in the connected element over the combined cross-section of the connected element and the fillers or shims;

(3) The size of the *joint* shall be increased to accommodate a number of bolts that is equivalent to the total number required in (2) above; or

(4) The *joint* shall be designed as a *slip-critical joint*. The slip resistance of the *joint* shall not be reduced for the presence of fillers or shims.

B5.2. Combined Shear and Tension Stress

When combined shear and tension loads are transmitted by an ASTM A325, A490 or F1852 bolt, the bolt shall be proportioned so that the tensile stress F_t, ksi, on the cross-sectional area based upon the nominal diameter of bolt A_b produced by forces applied to the connected parts, shall not exceed the values computed from the equations in Table B5.2, where f_v, the shear stress produced by the same forces, shall not exceed the value for shear determined in accordance with the requirements in Section B5.1.

Table B5.1. Allowable Stress in Bolts

Applied Load Condition		Allowable Stress F_a, ksi	
		ASTM A325 or F1852 Bolt	ASTM A490 Bolt
Tension [a]	Static	44	54
	Fatigue	See Section B5.5	
Shear [a,b]	Threads included in shear plane	21	28
	Threads excluded from shear plane	30	40

[a] Except as required in Section B5.2.
[b] In shear *connections* that transmit axial force and have length between extreme bolts measured parallel to the line of force exceeds 50 in., tabulated values shall be reduced by 20 percent.

Table B5.2. Allowable Tensile Stress F_t for Bolts Subject to Combined Shear and Tension

Thread Condition	Allowable Tensile Stress F_t, ksi	
	ASTM A325 or F1852 Bolt	ASTM A490 Bolt
Threads included in Shear plane	$\sqrt{(44)^2 - 4.39 f_v^2}$	$\sqrt{(54)^2 - 3.71 f_v^2}$
Threads excluded From shear plane	$\sqrt{(44)^2 - 2.15 f_v^2}$	$\sqrt{(54)^2 - 1.82 f_v^2}$

B5.3. Allowable Bearing at Bolt Holes

For *joints*, the allowable bearing strength shall be taken as the sum of the strengths of the connected material at the individual bolt holes.

The allowable bearing strength of the connected material at a standard bolt hole, oversized bolt hole, short-slotted bolt hole independent of the direction of loading or long-slotted bolt hole with the slot parallel to the direction of the bearing load is R_a, where:

(1) when deformation of the bolt hole at service load is a design consideration;

$$R_a = 0.6 L_c t F_u \leq 1.2 d_b t F_u \qquad \text{(Equation B5.2)}$$

(2) when deformation of the bolt hole at service load is not a design consideration;

$$R_a = 0.75 L_c t F_u \leq 1.5 d_b t F_u \qquad \text{(Equation B5.3)}$$

The allowable bearing strength of the connected material at a long-slotted bolt hole with the slot perpendicular to the direction of the bearing load is R_a, where:

$$R_a = 0.5 L_c t F_u \leq d_b t F_u \qquad \text{(Equation B5.4)}$$

In Equations B5.2, B5.3 and B5.4,

R_a = allowable bearing strength of the connected material, kips;
F_u = specified minimum tensile strength (per unit area) of the connected material, ksi;
L_c = clear distance, in the direction of load, between the edge of the hole and the edge of the adjacent hole or the edge of the material, in.;
d_b = nominal bolt diameter, in.; and,
t = thickness of the connected material, in.

B5.4. Allowable Slip Resistance

The allowable slip resistance is R_a, where:

$$R_a = H\mu D T_m N_b \left(1 - \frac{T}{D T_m N_b}\right) \qquad \text{(Equation B5.5)}$$

where

H = 1.0 for standard holes
= 0.85 for oversized and short-slotted holes
= 0.70 for long-slotted holes perpendicular to the direction of load
= 0.60 for long-slotted holes parallel to the direction of load;
μ = *mean slip coefficient* for Class A, B or C faying surfaces, as applicable, or as established by testing in accordance with Appendix A (see Section 3.2.2(b))

Table B5.3. Allowable Stress for Fatigue Loading

Number of Cycles	Maximum Bolt Stress for Design at Service Loads[a], ksi	
	ASTM A325 or F1852 Bolt	ASTM A490 Bolt
Not more than 20,000	44	54
From 20,000 to 500,000	40	49
More than 500,000	31	38

[a] Including the effects of *prying action*, if any, but excluding the pretension.

	=	0.33 for Class A *faying surfaces* (uncoated clean mill scale steel sur faces or surfaces with Class A coatings on blast cleaned steel)
	=	0.50 for Class B surfaces (uncoated blast-cleaned steel surfaces or surfaces with Class B coatings on blast-cleaned steel)
	=	0.35 for Class C surfaces (roughened hot-dip galvanized surfaces);
D	=	0.80, a slip probability factor that reflects the distribution of actual slip coefficient values about the mean, the ratio of measured bolt tensile strength to the specified minimum values, and a slip probability level; the use of other values of D shall be approved by the *Engineer of Record*;
T_m	=	specified minimum bolt pretension (for pretensioned joints as specified in Table 8.1), kips;
N_b	=	number of bolts in the joint; and,
T	=	applied service load in tension (tensile component of applied service load for combined shear and tension loading), kips
	=	zero if the joint is subject to shear only

B5.5. Tensile Fatigue

The tensile stress in the bolt that results from the cyclic application of externally applied service loads and the prying force, if any, but not the pretension, shall not exceed the stress in Table B5.3. The nominal diameter of the bolt shall be used in calculating the bolt stress. The connected parts shall be proportioned so that the calculated prying force does not exceed 30 percent of the externally applied load. *Joints* that are subject to tensile fatigue loading shall be pretensioned in accordance with Section 4.2 or slip-critical in accordance with Section 4**.**

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Code of Standard Practice for Steel Buildings and Bridges

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Prepared by the American Institute of Steel Construction, Inc. under the direction of the AISC Committee on the Code of Standard Practice and issued by the AISC Board of Directors.

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PREFACE

As in any industry, trade practices have developed among those that are involved in the design, purchase, fabrication and erection of structural steel. This Code provides a useful framework for a common understanding of the acceptable standards when contracting for structural steel. As such, it is useful for owners, architects, engineers, general contractors, construction managers, fabricators, steel detailers, erectors and others that are associated with construction in structural steel. Unless specific provisions to the contrary are contained in the contract documents, the existing trade practices that are contained herein are considered to be the standard custom and usage of the industry and are thereby incorporated into the relationships between the parties to a contract.

The Symbols and Glossary are an integral part of this Code. In many sections of this Code, a non-mandatory Commentary has been prepared to provide background and further explanation for the corresponding Code provisions. The user is encouraged to consult it.

Since the first edition of this Code was published in 1924, AISC has continuously surveyed the structural steel design community and construction industry to determine standard trade practices. Since then, this Code has been periodically updated to reflect new and changing technology and industry practices.

The 2000 edition was the fifth complete revision of this Code since it was first published. The 2005 edition is not a complete revision but does add several important changes and updates. It is the result of the deliberations of a fair and balanced Committee, the membership of which included six structural engineers, two architects, one code official, one general contractor, eight fabricators, one steel detailer, three erectors, two inspectors, and one attorney. The following changes have been made in this revision:

- The intent of Section 1.1 has been clarified with additional Commentary.
- Section 1.5.2 has been modified to better address Owner-established performance criteria.
- The intent of the first sentence in Section 1.8.2 has been clarified.
- The order of paragraphs in Section 3.3 has been reversed to highlight that discovered discrepancies must be reported for resolution.
- The requirements in Section 3.4 for scale of design drawings have been modified.
- A requirement has been added in Section 4.2 for identification of Shop and Erection Drawings. Additionally, a paragraph has been added in the Commentary to this section addressing the use of independent detailing services, and the paragraph addressing the submittal schedule has been modified.
- A paragraph has been added to the Commentary in Section 4.4 addressing Shop and Erection Drawings that are approved subject to corrections noted, as well as Shop and Erection Drawings that are not approved.

- Coverage has been added of the RFI process in Section 4.6. Concurrently, explicit mention of RFIs has been added in Sections 3.5 and 4.4.2. Additionally, definitions have been added in the Glossary of the terms RFI, Clarification and Revision.
- The requirements for material identification have been modified in Section 6.1. Compatible modifications have also been made in Section 5.1.1.
- The requirements in Section 6.4.5 have been expanded to address fabricated trusses specified without camber. Compatible additions have been made in Sections 7.13.1.2(g) and (h).
- Section 7.4 has been modified to change "building lines" to "lines".
- The Established Column Line definition in the Glossary has been changed, the definition of the term Column Line has been changed, and the usage of these terms in Section 7.5.1 has been changed for consistency with these definitions.
- Additional Commentary has been provided in Section 7.10.1 to illustrate the required description of the lateral load resisting system.
- Explicit mention of Erection Bracing Drawings has been added in the Commentary to Section 7.10.3.
- The intent of Section 8.5.5 has been clarified.
- Item 9.2.2(d) has been modified to change "detailed overall length" to "overall length".
- Appendix A has been added to explicitly allow the user of this Code to choose to use electronic means for the exchange of project information.

By the AISC Committee on the Code of Standard Practice,

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GLOSSARY

The following terms are used in this Code. Where used, they are capitalized to alert the user that the term is defined in this Glossary.

AASHTO. American Association of State Highway and Transportation Officials.

Adjustable Items. See Section 7.13.1.3.

AESS. See Architecturally Exposed Structural Steel.

AISC. American Institute of Steel Construction, Inc.

Anchor Bolt. See Anchor Rod.

Anchor Rod. A mechanical device that is either cast or drilled and chemically adhered, grouted or wedged into concrete and/or masonry for the purpose of the subsequent attachment of Structural Steel.

Anchor-Rod Group. A set of Anchor Rods that receives a single fabricated Structural Steel shipping piece.

ANSI. American National Standards Institute.

Architect. The entity that is professionally qualified and duly licensed to perform architectural services.

Architecturally Exposed Structural Steel. See Section 10.

AREMA. American Railway Engineering and Maintenance of Way Association.

ASME. American Society of Mechanical Engineers.

ASTM. American Society for Testing and Materials.

AWS. American Welding Society.

Bearing Devices. Shop-attached base and bearing plates, loose base and bearing plates and leveling devices, such as leveling plates, leveling nuts and washers and leveling screws.

CASE. Council of American Structural Engineers.

Clarification. An interpretation, of the Design Drawings or Specifications that have been Released for Construction, made in response to an RFI or a note on an approval drawing and providing an explanation that neither revises the information that has been Released for Construction nor alters the cost or schedule of performance of the work.

the Code, this Code. This document, the AISC *Code of Standard Practice for Steel Buildings and Bridges* as adopted by the American Institute of Steel Construction, Inc.

Column line. The grid line of column centers set in the field based on the dimensions shown on the structural design drawings and using the building layout provided by the Owners Designated Representative for Construction. Column offsets are taken from the column line. The column line may be straight or curved as shown in the structural design drawings.

Connection. An assembly of one or more joints that is used to transmit forces between two or more members and/or connection elements.

Contract Documents. The documents that define the responsibilities of the parties that are involved in bidding, fabricating and erecting Structural Steel. These documents normally include the Design Drawings, the Specifications and the contract.

Design Drawings. The graphic and pictorial portions of the Contract Documents showing the design, location and dimensions of the work. These documents generally include plans, elevations, sections, details, schedules, diagrams and notes.

Embedment Drawings. Drawings that show the location and placement of items that are installed to receive Structural Steel.

EOR. See Structural Engineer of Record.

Engineer. See Structural Engineer of Record.

Engineer of Record. See Structural Engineer of Record.

Erection Bracing Drawings. Drawings that are prepared by the Erector to illustrate the sequence of erection, any requirements for temporary supports and the requirements for raising, bolting and/or welding. These drawings are in addition to the Erection Drawings.

Erection Drawings. Field-installation or member-placement drawings that are prepared by the Fabricator to show the location and attachment of the individual shipping pieces.

Erector. The entity that is responsible for the erection of the Structural Steel.

Established Column Line. The actual field line that is most representative of the erected column centers along a line of columns placed using the dimensions shown in the structural Design Drawings and the lines and bench marks established by the Owner's Designated Representative for Construction, to be used in applying the erection tolerances given in this Code for column shipping pieces.

Fabricator. The entity that is responsible for fabricating the Structural Steel.

Hazardous Materials. Components, compounds or devices that are either encountered during the performance of the contract work or incorporated into it containing substances that, not withstanding the application of reasonable care, present a threat of harm to persons and/or the environment.

Inspector. The Owner's testing and inspection agency.

MBMA. Metal Building Manufacturers Association.

Mill Material. Steel mill products that are ordered expressly for the requirements of a specific project.

Owner. The entity that is identified as such in the Contract Documents.

Owner's Designated Representative for Construction. The Owner or the entity that is responsible to the Owner for the overall construction of the project, including its planning, quality and completion. This is usually the general contractor, the construction manager or similar authority at the job site.

Owner's Designated Representative for Design. The Owner or the entity that is responsible to the Owner for the overall structural design of the project, including the Structural Steel frame. This is usually the Structural Engineer of Record.

Plans. See Design Drawings.

RCSC. Research Council on Structural Connections.

Released for Construction. The term that describes the status of Contract Documents that are in such a condition that the Fabricator and the Erector can rely upon them for the performance of their work, including the ordering of material and the preparation of Shop and Erection Drawings.

Revision. An instruction or directive providing information that differs from information that has been Released for Construction. A Revision may, but does not always, impact the cost or schedule of performance of the work.

RFI. A written request for information or clarification generated during the construction phase of the project.

SER. See Structural Engineer of Record.

Shop Drawings. Drawings of the individual Structural Steel shipping pieces that are to be produced in the fabrication shop.

SJI. Steel Joist Institute.

Specifications. The portion of the Contract Documents that consists of the written requirements for materials, standards and workmanship.

SSPC. SSPC: The Society for Protective Coatings, which was formerly known as the Steel Structures Painting Council.

Standard Structural Shapes. Hot-rolled W-, S-, M- and HP-shapes, channels and angles listed in ASTM A6/A6M; structural tees split from the hot-rolled W-, S- and M-shapes listed in ASTM A6/A6M; hollow structural sections produced to ASTM A500, A501, A618 or A847; and, steel pipe produced to ASTM A53/A53M.

Steel Detailer. The entity that produces the Shop and Erection Drawings.

Structural Engineer of Record. The licensed professional who is responsible for sealing the Contract Documents, which indicates that he or she has performed or supervised the analysis, design and document preparation for the structure and has knowledge of the load-carrying structural system.

Structural Steel. The elements of the structural frame as given in Section 2.1.

Tier. The Structural Steel framing defined by a column shipping piece.

Weld Show-Through. In Architecturally Exposed Structural Steel, visual indication of the presence of a weld or welds on the side of the member opposite the weld.

CODE OF STANDARD PRACTICE
FOR STEEL BUILDINGS AND BRIDGES

SECTION 1. GENERAL PROVISIONS

1.1. Scope

In the absence of specific instructions to the contrary in the Contract Documents, the trade practices that are defined in this Code shall govern the fabrication and erection of Structural Steel.

Commentary:

The practices defined in this Code are the commonly accepted standards of custom and usage for Structural Steel fabrication and erection, which generally represent the most efficient approach. This Code is not intended to define a professional standard of care for the Owners Designated Representative for Design, change the duties and responsibilities of the Owner, Contractor, Architect or Structural Engineer from those set forth in the Contract Documents, or assign to the Owner, Architect or Structural Engineer any duty or authority to undertake responsibility inconsistent with the provisions of the Contract Documents.

This Code is not applicable to steel joists or metal building systems, which are addressed by SJI and MBMA, respectively.

1.2. Referenced Specifications, Codes and Standards

The following documents are referenced in this Code:

AASHTO Specification—The 2004 AASHTO *LRFD Bridge Design Specifications*, 3rd Edition, with interims, or the 2002 AASHTO *Standard Specifications for Highway Bridges*, 17th Edition, with interims.

AISC Manual of Steel Construction—The AISC *Manual of Steel Construction, 13th Edition.*

AISC Seismic Provisions—The AISC *Seismic Provisions for Structural Steel Buildings*, March 9, 2005.

AISC Specification—The AISC *Specification for Structural Steel Buildings*, March 9, 2005.

ANSI/ASME B46.1—ANSI/ASME B46.1-95, Surface Texture (Surface Roughness, Waviness and Lay).

AREMA Specification—The 1999 AREMA *Manual for Railway Engineering, Volume II—Structures, Chapter 15*.

ASTM A6/A6M—04a, *Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling.*

ASTM A53/A53M—02, *Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless*.

ASTM A325—04, *Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength*.

ASTM A325M—04, *Standard Specification for High-Strength Bolts for Structural Steel Joints (Metric)*.

ASTM A490—04, *Standard Specification for Heat-Treated Steel Structural Bolts, 150 ksi Minimum Tensile Strength*.

ASTMA490M—04, *Standard Specification for High-Strength Steel Bolts, Classes 10.9 and 10.9.3, for Structural Steel Joints (Metric)*.

ASTM A500—03a, *Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes*. No metric equivalent exists.

ASTM A501—01, *Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing*. No metric equivalent exists.

ASTM A618—04, *Standard Specification for Hot-Formed Welded and Seamless High-Strength Low-Alloy Structural Tubing*. No metric equivalent exists.

ASTM A847—99a(2003), *Standard Specification for Cold-Formed Welded and Seamless High-Strength, Low-Alloy Structural Tubing with Improved Atmospheric Corrosion Resistance*. No metric equivalent exists.

ASTM F1852/F1852M—04, *Standard Specification for "Twist-Off" Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength.*

AWS D1.1—The AWS D1.1 *Structural Welding Code—Steel*, 2004.

CASE Document 11—*An Agreement Between Structural Engineer of Record and Contractor for Transfer of Computer Aided Drafting (CAD) files on Electronic Media*, 2000

CASE Document 962—The *National Practice Guidelines for the Structural Engineer of Record*, Fourth Edition, 2000.

RCSC Specification—The *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, 2004.

SSPC SP2—SSPC *Surface Preparation Specification No. 2, Hand Tool Cleaning*, 2004.

SSPC SP6—SSPC *Surface Preparation Specification No. 6, Commercial Blast Cleaning*, 2004.

1.3. Units

In this Code, the values stated in either U.S. customary units or metric units shall be used. Each system shall be used independently of the other.

Commentary:
In this Code, dimensions, weights and other measures are given in U.S. customary units with rounded or rationalized metric-unit equivalents in

brackets. Because the values stated in each system are not exact equivalents, the selective combination of values from each of the two systems is not permitted.

1.4. Design Criteria

For buildings, in the absence of other design criteria, the provisions in the AISC Specification shall govern the design of the Structural Steel. For bridges, in the absence of other design criteria, the provisions in the AASHTO Specification and AREMA Specification shall govern the design of the Structural Steel, as applicable.

1.5. Responsibility for Design

1.5.1. When the Owner's Designated Representative for Design provides the design, Design Drawings and Specifications, the Fabricator and the Erector are not responsible for the suitability, adequacy or building-code conformance of the design.

1.5.2. When the Owner enters into a direct contract with the Fabricator to both design and fabricate an entire, completed steel structure, the Fabricator shall be responsible for the suitability, adequacy, conformance with Owner-established performance criteria, and building-code conformance of the Structural Steel design. The Owner shall be responsible for the suitability, adequacy and building-code conformance of the non-Structural Steel elements and shall establish the performance criteria for the Structural Steel frame.

1.6. Patents and Copyrights

The entity or entities that are responsible for the specification and/or selection of proprietary structural designs shall secure all intellectual property rights necessary for the use of those designs.

1.7. Existing Structures

1.7.1. Demolition and shoring of any part of an existing structure are not within the scope of work that is provided by either the Fabricator or the Erector. Such demolition and shoring shall be performed in a timely manner so as not to interfere with or delay the work of the Fabricator and the Erector.

1.7.2. Protection of an existing structure and its contents and equipment, so as to prevent damage from normal erection processes, is not within the scope of work that is provided by either the Fabricator or the Erector. Such protection shall be performed in a timely manner so as not to interfere with or delay the work of the Fabricator or the Erector.

1.7.3. Surveying or field dimensioning of an existing structure is not within the scope of work that is provided by either the Fabricator or the Erector. Such surveying or field dimensioning, which is necessary for the completion of Shop and Erection Drawings and fabrication, shall be performed and furnished to the Fabricator in a timely manner so as not to interfere with or delay the work of the Fabricator or the Erector.

1.7.4. Abatement or removal of Hazardous Materials is not within the scope of work that is provided by either the Fabricator or the Erector. Such abatement or removal shall be performed in a timely manner so as not to interfere with or delay the work of the Fabricator and the Erector.

1.8. Means, Methods and Safety of Erection

1.8.1. The Erector shall be responsible for the means, methods and safety of erection of the Structural Steel frame.

1.8.2. The Structural Engineer of Record shall be responsible for the structural adequacy of the design of the structure in the completed project. The Structural Engineer of Record shall not be responsible for the means, methods and safety of erection of the Structural Steel frame. See also Sections 3.1.4 and 7.10.

SECTION 2. CLASSIFICATION OF MATERIALS

2.1. Definition of Structural Steel

Structural Steel shall consist of the elements of the structural frame that are shown and sized in the structural Design Drawings, essential to support the design loads and described as:

Anchor Rods that will receive Structural Steel.
Base plates.
Beams, including built-up beams, if made from Standard Structural Shapes and/or plates.
Bearing plates.
Bearings of steel for girders, trusses or bridges.
Bracing, if permanent.
Canopy framing, if made from Standard Structural Shapes and/or plates.
Columns, including built-up columns, if made from Standard Structural Shapes and/or plates.
Connection materials for framing Structural Steel to Structural Steel.
Crane stops, if made from Standard Structural Shapes and/or plates.
Door frames, if made from Standard Structural Shapes and/or plates and if part of the Structural Steel frame.
Edge angles and plates, if attached to the Structural Steel frame or steel (open-web) joists.
Embedded Structural Steel parts, other than bearing plates, that will receive Structural Steel.
Expansion joints, if attached to the Structural Steel frame.
Fasteners for connecting Structural Steel items: permanent shop bolts, nuts and washers; shop bolts, nuts and washers for shipment; field bolts, nuts and washers for permanent Connections; and, permanent pins.
Floor-opening frames, if made from Standard Structural Shapes and/or plates and attached to the Structural Steel frame or steel (open-web) joists.
Floor plates (checkered or plain), if attached to the Structural Steel frame.
Girders, including built-up girders, if made from Standard Structural Shapes and/or plates.
Girts, if made from Standard Structural Shapes.
Grillage beams and girders.
Hangers, if made from Standard Structural Shapes, plates and/or rods and framing Structural Steel to Structural Steel.
Leveling nuts and washers.
Leveling plates.
Leveling screws.
Lintels, if attached to the Structural Steel frame.
Marquee framing, if made from Standard Structural Shapes and/or plates.

Machinery supports, if made from Standard Structural Shapes and/or plates and attached to the Structural Steel frame.
Monorail elements, if made from Standard Structural Shapes and/or plates and attached to the Structural Steel frame.
Posts, if part of the Structural Steel frame.
Purlins, if made from Standard Structural Shapes.
Relieving angles, if attached to the Structural Steel frame.
Roof-opening frames, if made from Standard Structural Shapes and/or plates and attached to the Structural Steel frame or steel (open-web) joists.
Roof-screen support frames, if made from Standard Structural Shapes.
Sag rods, if part of the Structural Steel frame and connecting Structural Steel to Structural Steel.
Shear stud connectors, if specified to be shop attached.
Shims, if permanent.
Struts, if permanent and part of the Structural Steel frame.
Tie rods, if part of the Structural Steel frame.
Trusses, if made from Standard Structural Shapes and/or built-up members.
Wall-opening frames, if made from Standard Structural Shapes and/or plates and attached to the Structural Steel frame.
Wedges, if permanent.

Commentary:
The Fabricator normally fabricates the items listed in Section 2.1. Such items must be shown, sized and described in the structural Design Drawings. Bracing includes vertical bracing for resistance to wind and seismic load and structural stability, horizontal bracing for floor and roof systems and permanent stability bracing for components of the Structural Steel frame.

2.2. Other Steel, Iron or Metal Items

Structural Steel shall not include other steel, iron or metal items that are not generally described in Section 2.1, even where such items are shown in the structural Design Drawings or are attached to the Structural Steel frame. Other steel, iron or metal items include but are not limited to:

Bearings, if non-steel.
Cables for permanent bracing or suspension systems.
Castings.
Catwalks.
Chutes.
Cold-formed steel products.
Cold-rolled steel products, except those that are specifically covered in the AISC Specification.
Corner guards.

Crane rails, splices, bolts and clamps.
Crane stops, if not made from Standard Structural Shapes or plates.
Door guards.
Embedded steel parts, other than bearing plates, that do not receive Structural Steel or that are embedded in precast concrete.
Expansion joints, if not attached to the Structural Steel frame.
Flagpole support steel.
Floor plates (checkered or plain), if not attached to the Structural Steel frame.
Forgings.
Gage-metal products.
Grating.
Handrail.
Hangers, if not made from Standard Structural Shapes, plates and/or rods or not framing Structural Steel to Structural Steel.
Hoppers.
Items that are required for the assembly or erection of materials that are furnished by trades other than the Fabricator or Erector.
Ladders.
Lintels, if not attached to the Structural Steel frame.
Masonry anchors.
Miscellaneous metal.
Ornamental metal framing.
Pressure vessels.
Reinforcing steel for concrete or masonry.
Relieving angles, if not attached to the Structural Steel frame.
Roof screen support frames, if not made from Standard Structural Shapes.
Safety cages.
Shear stud connectors, if specified to be field installed.
Stacks.
Stairs.
Steel deck.
Steel (open-web) joists.
Steel joist girders.
Tanks.
Toe plates.
Trench or pit covers.

Commentary:
Section 2.2 includes many items that may be furnished by the Fabricator if contracted to do so by specific notation and detail in the Contract Documents. When such items are contracted to be provided by the Fabricator, coordination will normally be required between the Fabricator and other material suppliers

and trades. The provisions in this Code are not intended to apply to items in Section 2.2.

In previous editions of this Code, provisions regarding who should normally furnish field-installed shear stud connectors and cold-formed steel deck support angles were included in Section 7.8. These provisions have been eliminated since field-installed shear stud connectors and steel deck support angles are not defined as Structural Steel in this Code.

SECTION 3. DESIGN DRAWINGS AND SPECIFICATIONS

3.1. Structural Design Drawings and Specifications

Unless otherwise indicated in the Contract Documents, the structural Design Drawings shall be based upon consideration of the design loads and forces to be resisted by the Structural Steel frame in the completed project.

The structural Design Drawings shall clearly show the work that is to be performed and shall give the following information with sufficient dimensions to accurately convey the quantity and nature of the Structural Steel to be fabricated:

(a) The size, section, material grade and location of all members;
(b) All geometry and working points necessary for layout;
(c) Floor elevations;
(d) Column centers and offsets;
(e) The camber requirements for members; and,
(f) The information that is required in Sections 3.1.1 through 3.1.6.

The Structural Steel Specification shall include any special requirements for the fabrication and erection of the Structural Steel.

The structural Design Drawings, Specifications and addenda shall be numbered and dated for the purposes of identification.

Commentary:
Contract Documents vary greatly in complexity and completeness. Nonetheless, the Fabricator and the Erector must be able to rely upon the accuracy and completeness of the Contract Documents. This allows the Fabricator and the Erector to provide the Owner with bids that are adequate and complete. It also enables the preparation of the Shop and Erection Drawings, the ordering of materials and the timely fabrication and erection of shipping pieces.

In some cases, the Owner can benefit when reasonable latitude is allowed in the Contract Documents for alternatives that can reduce cost without compromising quality. However, critical requirements that are necessary to protect the Owner's interest, that affect the integrity of the structure or that are necessary for the Fabricator and the Erector to proceed with their work must be included in the Contract Documents. Some examples of critical information include:

Standard specifications and codes that govern Structural Steel design and construction, including bolting and welding.
Material specifications.
Special material requirements to be reported on the certified mill test reports.
Welded-joint configuration.

Weld-procedure qualification.
Special requirements for work of other trades.
Final disposition of backing bars and runoff tabs.
Lateral bracing.
Stability bracing.
Connections or data for Connection selection and/or completion.
Restrictions on Connection types.
Column stiffeners (also known as continuity plates).
Column web doubler plates.
Bearing stiffeners on beams and girders.
Web reinforcement.
Openings for other trades.
Surface preparation and shop painting requirements.
Shop and field inspection requirements.
Non-destructive testing requirements, including acceptance criteria.
Special requirements on delivery.
Special erection limitations.
Identification of non-Structural Steel elements that interact with the Structural Steel frame to provide for the lateral stability of the Structural Steel frame (see Section 3.1.4).
Column differential shortening information.
Special fabrication and erection tolerances for AESS.
Special pay-weight provisions.

3.1.1. Permanent bracing, column stiffeners, column web doubler plates, bearing stiffeners in beams and girders, web reinforcement, openings for other trades and other special details, where required, shall be shown in sufficient detail in the structural Design Drawings so that the quantity, detailing and fabrication requirements for these items can be readily understood.

3.1.2. The Owner's Designated Representative for Design shall either show the complete design of the Connections in the structural Design Drawings or allow the Fabricator to select or complete the Connection details while preparing the Shop and Erection Drawings. When the Fabricator is allowed to select or complete the Connection details, the following information shall be provided in the structural Design Drawings:

(a) Any restrictions on the types of Connections that are permitted;
(b) Data concerning the loads, including shears, moments, axial forces and transfer forces, that are to be resisted by the individual members and their Connections, sufficient to allow the Fabricator to select or complete the Connection details while preparing the Shop and Erection Drawings;
(c) Whether the data required in (b) is given at the service-load level or the factored-load level; and,

(d) Whether LRFD or ASD is to be used in the selection or completion of Connection details.

When the Fabricator selects or completes the Connection details, the Fabricator shall utilize the requirements in the AISC Specification and the Contract Documents and submit the Connection details to the Owner's Designated Representative for Design for approval.

Commentary:
When the Owner's Designated Representative for Design shows the complete design of the Connections in the structural Design Drawings, the following information is included:

(a) All weld sizes and lengths;
(b) All bolt sizes, locations, quantities and grades;
(c) All plate and angle sizes, thicknesses and dimensions; and,
(d) All work point locations and related information.

The intent of this approach is that complete information necessary for Connection detailing, fabrication and erection is shown in the structural Design Drawings. The Steel Detailer will then be able to transfer this information to the Shop and Erection Drawings, applying it to the individual pieces being detailed.

When the Owner's Designated Representative for Design allows the Fabricator to select or complete the Connections, this is commonly done by referring to tables in the Contract Documents or in the AISC Manual of Steel Construction, or by schematically showing the types of Connections required in the structural Design Drawings. The Steel Detailer will then configure the Connections based upon the design loads and other information given in the structural Design Drawings. If the desired Connection is not covered in those tables, a detail of the "special" Connection should be contained in the structural Design Drawings. This detail should provide such information as weld sizes, plate thicknesses and quantities of bolts. However, there may be some geometry and dimensional information that the Steel Detailer must develop. The intent of this method is that the Steel Detailer will select the Connection materials and configuration from the referenced tables or complete the specific Connection configuration (i.e. dimensions, edge distances and bolt spacing) based upon the Connection details that are shown in the structural Design Drawings.

This method will require the skill of an experienced Steel Detailer, who is familiar with the AISC requirements for Connection configurations, capable and experienced in the use of the Connection tables in the AISC Manual of Steel Construction and capable of calculating dimensions and adapting a typical Connection detail to similar situations. Notations of loadings in the structural Design Drawings are only to facilitate selection of the Connections from the referenced tables. It is not the intent of this method that the Steel Detailer practice engineering.

If there are any restrictions as to the types of Connections to be used, particularly as it relates to simple shear Connections, it is required that these limitations be set forth in the structural Design Drawings and Specifications. There are a variety of Connections available in the AISC Manual of Steel Construction for a given situation. Preference for a particular type will vary between Fabricators and Erectors. Stating these limitations, if any, in the structural Design Drawings and Specifications will help to avoid repeated changes to the Shop and Erection Drawings due to the selection of a Connection that is not acceptable to the Owner's Designated Representative for Design, thereby avoiding additional cost and/or delay for the redrawing of the Shop and Erection Drawings.

The structural Design Drawings must indicate the method of design used as LRFD or ASD. In order to conform to the spirit of the AISC Specification, the Connections must be selected using the same method and the corresponding references.

3.1.3. When leveling plates are to be furnished as part of the contract requirements, their locations and required thickness and sizes shall be specified in the Contract Documents.

3.1.4. When the Structural Steel frame, in the completely erected and fully connected state, requires interaction with non-Structural Steel elements (see Section 2) for strength and/or stability, those non-Structural Steel elements shall be identified in the Contract Documents as required in Section 7.10.

Commentary:
Examples of non-Structural Steel elements include diaphragms made of steel deck, diaphragms made of concrete on steel deck and masonry and/or concrete shear walls.

3.1.5. When camber is required, the magnitude, direction and location of camber shall be specified in the structural Design Drawings.

Commentary:
For cantilevers, the specified camber may be up or down, depending upon the framing and loading.

3.1.6. Specific members or portions thereof that are to be left unpainted shall be identified in the Contract Documents. When shop painting is required, the painting requirements shall be specified in the Contract Documents, including the following information:

(a) The identification of specific members or portions thereof to be painted;
(b) The surface preparation that is required for these members;
(c) The paint specifications and manufacturer's product identification that are required for these members; and,
(d) The minimum dry-film shop-coat thickness that is required for these members.

Commentary:
Some members or portions thereof may be required to be left unpainted, such as those that will be in contact and acting compositely with concrete, or those that will receive spray-applied fire protection materials.

3.2. Architectural, Electrical and Mechanical Design Drawings and Specifications

All requirements for the quantities, sizes and locations of Structural Steel shall be shown or noted in the structural Design Drawings. The use of architectural, electrical and/or mechanical Design Drawings as a supplement to the structural Design Drawings is permitted for the purposes of defining detail configurations and construction information.

3.3. Discrepancies

When discrepancies exist between the Design Drawings and Specifications, the Design Drawings shall govern. When discrepancies exist between scale dimensions in the Design Drawings and the figures written in them, the figures shall govern. When discrepancies exist between the structural Design Drawings and the architectural, electrical or mechanical Design Drawings or Design Drawings for other trades, the structural Design Drawings shall govern.

When a discrepancy is discovered in the Contract Documents in the course of the Fabricator's work, the Fabricator shall promptly notify the Owner's Designated Representative for Construction so that the discrepancy can be resolved by the Owner's Designated Representative for Design. Such resolution shall be timely so as not to delay the Fabricator's work. See Sections 3.5 and 9.3.

Commentary:
While it is the Fabricator's responsibility to report any discrepancies that are discovered in the Contract Documents, it is not the Fabricator's responsibility to discover discrepancies, including those that are associated with the coordination of the various design disciplines. The quality of the Contract Documents is the responsibility of the entities that produce those documents.

3.4. Legibility of Design Drawings

Design Drawings shall be clearly legible and drawn to an identified scale that is appropriate to clearly convey the information.

Commentary:
Historically, the most commonly accepted scale for structural steel plans has been 1/8 in. per ft [10 mm per 1 000 mm]. There are, however, situations where a smaller or larger scale is appropriate. Ultimately, consideration must be given to the clarity of the drawing.

The scaling of the Design Drawings to determine dimensions is not an accepted practice for detailing the Shop and Erection Drawings. However, it should be remembered when preparing Design Drawings that scaling may be the only method available when early-submission drawings are used to determine dimensions for estimating and bidding purposes.

3.5. Revisions to the Design Drawings and Specifications

Revisions to the Design Drawings and Specifications shall be made either by issuing new Design Drawings and Specifications or by reissuing the existing Design Drawings and Specifications. In either case, all Revisions, including Revisions that are communicated through responses to RFIs or the annotation of Shop and/or Erection Drawings (see Section 4.4.2), shall be clearly and individually indicated in the Contract Documents. The Contract Documents shall be dated and identified by Revision number. Each Design Drawing shall be identified by the same drawing number throughout the duration of the project, regardless of the Revision. See also Section 9.3.

Commentary:
Revisions to the Design Drawings and Specifications can be made by issuing sketches and supplemental information separate from the Design Drawings and Specifications. These sketches and supplemental information become amendments to the Design Drawings and Specifications and are considered new Contract Documents. All sketches and supplemental information must be uniquely identified with a number and date as the latest instructions until such time as they may be superseded by new information.

When revisions are made by revising and re-issuing the existing structural Design Drawings and/or Specifications, a unique revision number and date must be added to those documents to identify that information as the latest instructions until such time as they may be superseded by new information. The same unique drawing number must identify each Design Drawing throughout the duration of the project so that revisions can be properly tracked, thus avoiding confusion and miscommunication among the various entities involved in the project.

When revisions are communicated through the annotation of Shop or Erection Drawings or contractor submissions, such changes must be confirmed in writing by one of the aforementioned methods. This written confirmation is imperative to maintain control of the cost and schedule of a project and to avoid potential errors in fabrication.

3.6. Fast-Track Project Delivery

When the fast-track project delivery system is selected, release of the structural Design Drawings and Specifications shall constitute a Release for Construction, regardless of the status of the architectural, electrical, mechanical and other interfacing designs and Contract Documents. Subsequent revisions, if any, shall be the responsibility of the Owner and shall be made in accordance with Sections 3.5 and 9.3.

Commentary:
The fast-track project delivery system generally provides for a condensed schedule for the design and construction of a project. Under this delivery system, the Owner elects to Release for Construction the structural Design Drawings and Specifications, which may be partially complete, at a time that may precede the completion of and coordination with architectural, mechanical, electrical and other design work and Contract Documents. The release of these structural Design Drawings and Specifications may also precede the release of the General Conditions and Division 1 Specifications.

Release of the structural Design Drawings and Specifications to the Fabricator for ordering of material constitutes a Release for Construction. Accordingly, the Fabricator and the Erector may begin their work based upon those partially complete documents. As the architectural, mechanical, electrical and other design elements of the project are completed, revisions may be required in design and/or construction. Thus, when considering the fast-track project delivery system, the Owner should balance the potential benefits to the project schedule with the project cost contingency that may be required to allow for these subsequent revisions.

SECTION 4. SHOP AND ERECTION DRAWINGS

4.1. Owner Responsibility

The Owner shall furnish, in a timely manner and in accordance with the Contract Documents, complete structural Design Drawings and Specifications that have been Released for Construction. Unless otherwise noted, Design Drawings that are provided as part of a contract bid package shall constitute authorization by the Owner that the Design Drawings are Released for Construction

Commentary:
When the Owner issues Released-for-Construction Design Drawings and Specifications, the Fabricator and the Erector rely on the fact that these are the Owner's requirements for the project. This release is required by the Fabricator prior to the ordering of material and the preparation and completion of Shop and Erection Drawings.

To ensure the orderly flow of material procurement, detailing, fabrication and erection activities, on phased construction projects, it is essential that designs are not continuously revised after they have been Released for Construction. In essence, once a portion of a design is Released for Construction, the essential elements of that design should be "frozen" to ensure adherence to the contract price and construction schedule. Alternatively, all parties should reach a common understanding of the effects of future changes, if any, as they affect scheduled deliveries and added costs.

4.2. Fabricator Responsibility

Except as provided in Section 4.5, the Fabricator shall produce Shop and Erection Drawings for the fabrication and erection of the Structural Steel and is responsible for the following:

(a) The transfer of information from the Contract Documents into accurate and complete Shop and Erection Drawings; and,
(b) The development of accurate, detailed dimensional information to provide for the fit-up of parts in the field.

Each Shop and Erection Drawing shall be identified by the same drawing number throughout the duration of the project and shall be identified by revision number and date, with each specific revision clearly identified.

When the Fabricator submits a request to change Connection details that are described in the Contract Documents, the Fabricator shall notify the Owner's Designated Representatives for Design and Construction in writing in advance of the submission of the Shop and Erection Drawings. The Owner's Designated Representative for Design shall review and approve or reject the request in a timely manner.

When requested to do so by the Owner’s Designated Representative for Design, the Fabricator shall provide to the Owner’s Designated Representatives for Design and Construction its schedule for the submittal of Shop and Erection Drawings so as to facilitate the timely flow of information between all parties.

Commentary:
The fabricator is permitted to use the services of independent detailers to produce shop and erection drawings and to perform other support services such as producing advanced bills of material and bolt summaries.

As the Fabricator develops the detailed dimensional information for production of the Shop and Erection Drawings, there may be discrepancies, missing information or conflicts discovered in the Contract Documents. See Section 3.3.

When the Fabricator intends to make a submission of alternative Connection details to those shown in the Contract Documents, the Fabricator must notify the Owner’s Designated Representatives for Design and Construction in advance. This will allow the parties involved to plan for the increased effort that may be required to review the alternative Connection details. In addition, the Owner will be able to evaluate the potential for cost savings and/or schedule improvements against the additional design cost for review of the alternative Connection details by the Owner’s Designated Representative for Design. This evaluation by the Owner may result in the rejection of the alternative Connection details or acceptance of the submission for review based upon cost savings, schedule improvements and/or job efficiencies.

The Owner’s Designated Representative for Design may request the Fabricator’s schedule for the submittal of shop and erection drawings. This process is intended to allow the parties to plan for the staffing demands of the submission schedule. The Contract Documents may address this issue in more detail. In the absence of the requirement to provide this schedule, none need be provided.

When the Fabricator provides a schedule for the submission of the Shop and Erection Drawings, it must be recognized that this schedule may be affected by revisions and the response time to requests for missing information or the resolution of discrepancies.

4.3. Use of CAD Files and/or Copies of Design Drawings

The Fabricator shall neither use nor reproduce any part of the Design Drawings as part of the Shop or Erection Drawings without the written permission of the Owner's Designated Representative for Design. When CAD files or copies of the Design Drawings are made available for the Fabricator’s use, the Fabricator shall accept this information under the following conditions:

(a) All information contained in the CAD files or copies of the Design Drawings shall be considered instruments of service of the Owner's Designated Representative for Design and shall not be used for other projects, additions to the project or the completion of the project by others. CAD files and copies of the Design Drawings shall remain the property of the Owner's Designated Representative for Design and in no case shall the transfer of these CAD files or copies of the Design Drawings be considered a sale.
(b) The CAD files or copies of the Design Drawings shall not be considered to be Contract Documents. In the event of a conflict between the Design Drawings and the CAD files or copies thereof, the Design Drawings shall govern;
(c) The use of CAD files or copies of the Design Drawings shall not in any way obviate the Fabricator's responsibility for proper checking and coordination of dimensions, details, member sizes and fit-up and quantities of materials as required to facilitate the preparation of Shop and Erection Drawings that are complete and accurate as required in Section 4.2; and,
(d) The Fabricator shall remove information that is not required for the fabrication or erection of the Structural Steel from the CAD files or copies of the Design Drawings.

Commentary:
With the advent of electronic media and the internet, electronic copies of Design Drawings are becoming readily available to the Fabricator. As a result, the Owner's Designated Representative for Design may have reduced control over the unauthorized use of the Design Drawings. There are many copyright and other legal issues to be considered.

The Owner's Designated Representative for Design may choose to make CAD files or copies of the Design Drawings available to the Fabricator, and may charge a service or licensing fee for this convenience. In doing so, a carefully negotiated agreement should be established to set out the specific responsibilities of both parties in view of the liabilities involved for both parties. For a sample contract, see CASE Document 11.

The CAD files and/or copies of the Design Drawings are provided to the Fabricator for convenience only. The information therein should be adapted for use only in reference to the placement of Structural Steel members during erection. The Fabricator should treat this information as if it were fully produced by the Fabricator and undertake the same level of checking and quality assurance. When amendments or revisions are made to the Contract Documents, the Fabricator must update this reference material.

When CAD files or copies of the Design Drawings are provided to the Fabricator, they often contain other information, such as architectural backgrounds or references to other Contract Documents. This additional material should be removed when producing Shop and Erection Drawings to avoid the potential for confusion.

4.4. Approval

Except as provided in Section 4.5, the Shop and Erection Drawings shall be submitted to the Owner's Designated Representatives for Design and Construction for review and approval. These drawings shall be returned to the Fabricator within 14 calendar days. Approved Shop and Erection Drawings shall be individually annotated by the Owner's Designated Representatives for Design and Construction as either approved or approved subject to corrections noted. When so required, the Fabricator shall subsequently make the corrections noted and furnish corrected Shop and Erection Drawings to the Owner's Designated Representatives for Design and Construction.

Commentary:
As used in this Code, the 14-day allotment for the return of Shop and Erection Drawings is intended to represent the Fabricator's portal-to-portal time. The intent in this Code is that, in the absence of information to the contrary in the Contract Documents, 14 days may be assumed for the purposes of bidding, contracting and scheduling. A submittal schedule is commonly used to facilitate the approval process.

If a Shop or Erection Drawing is approved subject to corrections noted, the Owner's Designated Representative for Design may or may not require that it be re-submitted for record purposes following correction. If a Shop or Erection Drawing is not approved, revisions must be made and the drawing re-submitted until approval is achieved.

4.4.1. Approval of the Shop and Erection Drawings, approval subject to corrections noted and similar approvals shall constitute the following:

(a) Confirmation that the Fabricator has correctly interpreted the Contract Documents in the preparation of those submittals;
(b) Confirmation that the Owner's Designated Representative for Design has reviewed and approved the Connection details shown on the Shop and Erection Drawings and submitted in accordance with Section 3.1.2, if applicable; and,
(c) Release by the Owner's Designated Representatives for Design and Construction for the Fabricator to begin fabrication using the approved submittals.

Such approval shall not relieve the Fabricator of the responsibility for either the accuracy of the detailed dimensions in the Shop and Erection Drawings or the general fit-up of parts that are to be assembled in the field.

The Fabricator shall determine the fabrication schedule that is necessary to meet the requirements of the contract.

Commentary:
When considering the current language in this Section, the Committee sought language that would parallel the practices of CASE. In CASE Document 962, CASE indicates that when the design of some element of the primary structural system is left to someone other than the Structural Engineer of Record, "…such elements, including connections designed by others, should be reviewed by the Structural Engineer of Record. He [or she] should review such designs and details, accept or reject them and be responsible for their effects on the primary structural system." Historically, this Code has embraced this same concept.

From the inception of this Code, AISC and the industry in general have recognized that only the Owner's Designated Representative for Design has all the information necessary to evaluate the total impact of Connection details on the overall structural design of the project. This authority has traditionally been exercised during the approval process for Shop and Erection Drawings. The Owner's Designated Representative for Design has thus retained responsibility for the adequacy and safety of the entire structure since at least the 1927 edition of this Code.

4.4.2. Unless otherwise noted, any additions, deletions or Revisions that are indicated in responses to RFIs or on the approved Shop and Erection Drawings shall constitute authorization by the Owner that the additions, deletions or revisions are Released for Construction. The Fabricator and the Erector shall promptly notify the Owner's Designated Representative for Construction when any direction or notation in responses to RFIs or on the Shop or Erection Drawings or other information will result in an additional cost and/or a delay. See Sections 3.5 and 9.3.

Commentary:
When the Fabricator notifies the Owner's Designated Representative for Construction that a direction or notation in responses to RFIs or on the Shop or Erection Drawings will result in an additional cost or a delay, it is then normally the responsibility of the Owner's Designated Representative for Construction to subsequently notify the Owner's Designated Representative for Design.

4.5. Shop and/or Erection Drawings Not Furnished by the Fabricator

When the Shop and Erection Drawings are not prepared by the Fabricator, but are furnished by others, they shall be delivered to the Fabricator in a timely manner. These Shop and Erection Drawings shall be prepared, insofar as is practical, in accordance with the shop fabrication and detailing standards of the Fabricator. The Fabricator shall neither be responsible for the completeness or accuracy of Shop and Erection Drawings so furnished, nor for the general fit-up of the members that are fabricated from them.

4.6. The RFI Process

When Requests for Information (RFIs) are issued, the process shall include the maintenance of a written record of inquiries and responses related to interpretation and implementation of the Contract Documents, including the Clarifications and/or Revisions to the Contract Documents that result, if any. RFIs shall not be used for the incremental Release for Construction of Design Drawings. When RFIs involve discrepancies or Revisions, see Sections 3.3, 3.5, and 4.4.2.

Commentary:

The RFI process is most commonly used during the detailing process, but can also be used to forward inquiries by the Erector or to inform the Owners Designated Representative For Design in the event of a fabricator or erector error and to develop corrective measures to resolve such errors.

The RFI process is intended to provide a written record of inquiries and associated responses but not to replace all verbal communication between the parties on the project. RFIs should be prepared and responded to in a timely fashion so as not to delay the work of the Detailer, Fabricator, and Erector. Discussion of the RFI issues and possible solutions between the Fabricator, Erector, and Owner's Designated Representatives for Design and Construction often can facilitate timely and practical resolution. Unlike Shop and Erection Drawing submittals in Section 4.2, RFI response time can vary depending on the urgency of the issue, the amount of work required by the Owner's Designated Representatives for Design and Construction to develop a complete response, and other circumstances such as building official approval.

RFIs should be prepared in a standardized format, including RFI number and date, identity of the author, reference to a specific Design Drawing number (and specific detail as applicable) or Specification section, the needed response date, a description of a suggested solution (graphic depictions are recommended for more complex issues), and an indication of possible schedule and cost impacts. RFIs should be limited to one question each (unless multiple questions are interrelated to the same issue) to facilitate the resolution and minimize response time. Questions and proposed solutions presented in RFIs should be clear and complete. RFI responses should be equally clear and complete in the depictions of the solutions, and signed and dated by the responding party.

Unless otherwise noted, the Fabricator/Erector can assume that a response to an RFI constitutes a Release for Construction. However, if the response will result in an increase in cost or a delay in schedule, Section 4.4.2 requires that the Fabricator/Erector promptly inform the Owner's Designated Representatives for Design and Construction.

SECTION 5. MATERIALS

5.1. Mill Materials

Unless otherwise noted in the Contract Documents, the Fabricator is permitted to order the materials that are necessary for fabrication when the Fabricator receives Contract Documents that have been Released for Construction.

Commentary:
The Fabricator may purchase materials in stock lengths, exact lengths or multiples of exact lengths to suit the dimensions shown in the structural Design Drawings. Such purchases will normally be job-specific in nature and may not suitable for use on other projects or returned for full credit if subsequent design changes make these materials unsuitable for their originally intended use. The Fabricator should be paid for these materials upon delivery from the mill, subject to appropriate additional payment or credit if subsequent unanticipated modification or reorder is required. Purchasing materials to exact lengths is not considered fabrication.

5.1.1. Unless otherwise specified by means of special testing requirements in the Contract Documents, mill testing shall be limited to those tests that are required for the material in the ASTM specifications indicated in the Contract Documents. Materials ordered to special material requirements shall be marked by the supplier as specified in ASTM A6/A6M Section 12 prior to delivery to the Fabricator's shop or other point of use. Such material not so marked by the supplier, shall not be used until:

(a) Its identification is established by means of testing in accordance with the applicable ASTM specifications; and,
(b) A Fabricator's identification mark, as described in Section 6.1.2 and 6.1.3, has been applied.

5.1.2. When Mill Material does not satisfy ASTM A6/A6M tolerances for camber, profile, flatness or sweep, the Fabricator shall be permitted to perform corrective procedures, including the use of controlled heating and/or mechanical straightening, subject to the limitations in the AISC Specification.

Commentary:
Mill dimensional tolerances are completely set forth in ASTM A6/A6M. Normal variations in the cross-sectional geometry of Standard Structural Shapes must be recognized by the designer, the Fabricator, the Steel Detailer and the Erector (for example, see Figure C–5.1). Such tolerances are mandatory because roll wear, thermal distortions of the hot cross-section immediately after leaving the forming rolls and differential cooling distortions that take place on the cooling beds are all unavoidable. Geometric perfection of the cross-section is

not necessary for either structural or architectural reasons, if the tolerances are recognized and provided for.

ASTM A6/A6M also stipulates tolerances for straightness that are adequate for typical construction. However, these characteristics may be controlled or corrected to closer tolerances during the fabrication process when the added cost is justified by the special requirements for an atypical project.

5.1.3. When variations that exceed ASTM A6/A6M tolerances are discovered or occur after the receipt of Mill Material the Fabricator shall, at the Fabricator's option, be permitted to perform the ASTM A6/A6M corrective procedures for mill reconditioning of the surface of Structural Steel shapes and plates.

5.1.4. When special tolerances that are more restrictive than those in ASTM A6/A6M are required for Mill Materials, such special tolerances shall be specified in the Contract Documents. The Fabricator shall, at the Fabricator's option, be permitted to order material to ASTM A6/A6M tolerances and subsequently perform the corrective procedures described in Sections 5.1.2 and 5.1.3.

5.2. Stock Materials

5.2.1. If used for structural purposes, materials that are taken from stock by the Fabricator shall be of a quality that is at least equal to that required in the ASTM specifications indicated in the Contract Documents.

5.2.2. Certified mill test reports shall be accepted as sufficient record of the quality of materials taken from stock by the Fabricator. The Fabricator shall review and retain the certified mill test reports that cover such stock materials. However, the Fabricator need not maintain records that identify individual pieces of stock material against individual certified mill test reports, provided the Fabricator purchases stock materials that meet the requirements for material grade and quality in the applicable ASTM specifications.

5.2.3. Stock materials that are purchased under no particular specification, under a specification that is less rigorous than the applicable ASTM specifications or without certified mill test reports or other recognized test reports shall not be used without the approval of the Owner's Designated Representative for Design.

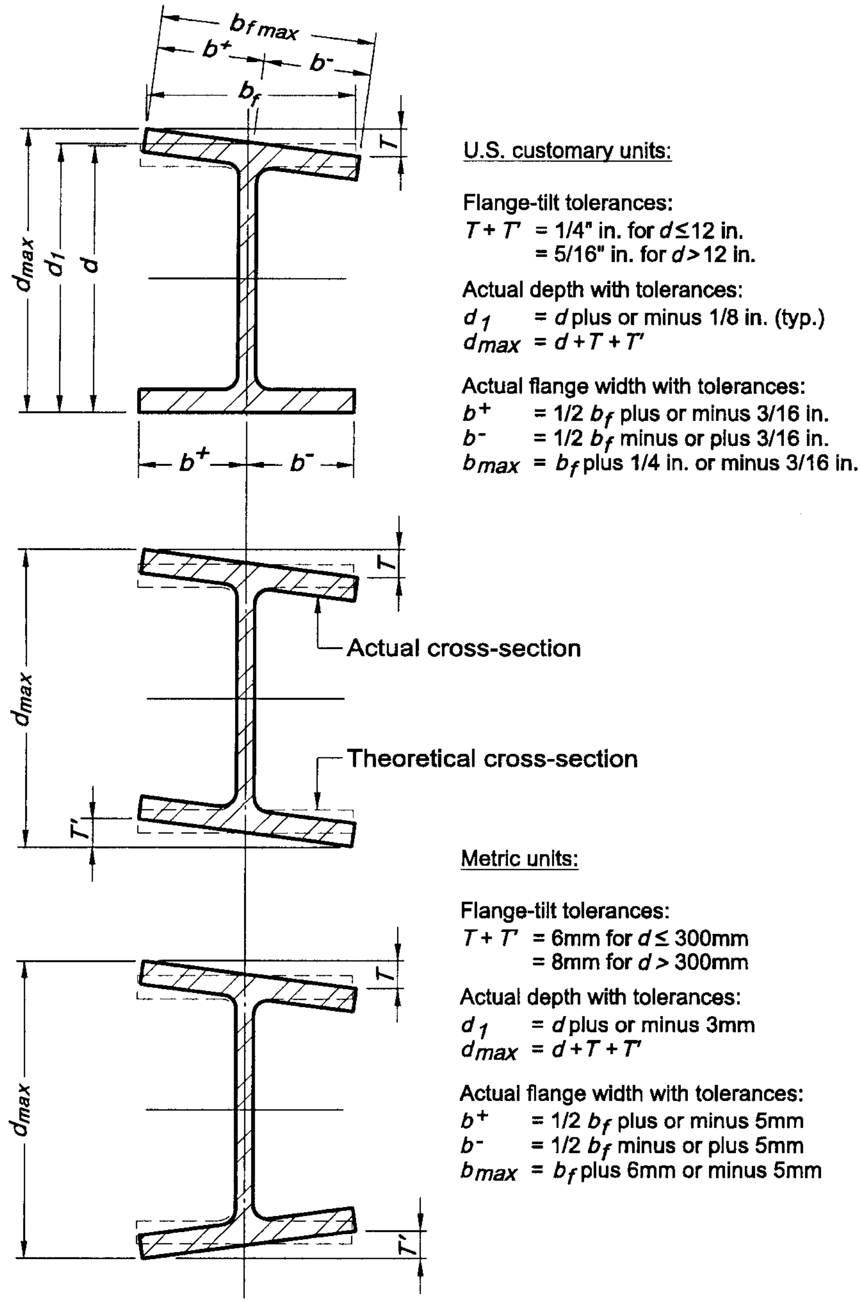


Figure C-5.1. Mill tolerances on the cross-section of a W-shape.

SECTION 6. SHOP FABRICATION AND DELIVERY

6.1. Identification of Material

6.1.1. The Fabricator shall be able to demonstrate by written procedure and actual practice a method of material identification, visible up to the point of assembling members as follows:

(a) For shop-standard material, identification capability shall include shape designation. Representative mill test reports shall be furnished by the Fabricator if requested to do so by the Owner's Designated Representative for Design, either in the Contract Documents or in separate written instructions given to the Fabricator prior to ordering Mill Materials.
(b) For material of grade other than shop-standard material, identification capability shall include shape designation and material grade. Representative mill test reports shall be furnished by the Fabricator if requested to do so by the Owner's Designated Representative for Design, either in the Contract Documents or in separate written instructions given to the Fabricator prior to ordering Mill Materials.
(c) For material ordered in accordance with an ASTM supplement or other special material requirements in the Contract Documents, identification capability shall include shape designation, material grade, and heat number. The corresponding mill test reports shall be furnished by the Fabricator if requested to do so by the Owner's Designated Representative for Design, either in the Contract Documents or in separate written instructions given to the Fabricator prior to ordering Mill Materials.

Unless an alternative system is established in the Fabricator's written procedures, shop-standard material shall be as follows:

Material	**Shop-standard material grade**
W and WT	ASTM A992
M, S, MT and ST	ASTM A36
HP	ASTM A36
L	ASTM A36
C and MC	ASTM A36
HSS	ASTM A500 grade B
Steel Pipe	ASTM A53 grade B
Plates and Bars	ASTM A36

Commentary:
The requirements in Section 6.1.1(a) will suffice for most projects. When material is of a strength level that differs from the shop-standard grade, the requirements in Section 6.1.1(b) apply. When special material requirements

apply, such as ASTM A6/A6M supplement S5 or S30 for CVN testing, ASTM A6/A6M supplement S8 for ultrasonic testing, or ASTM A588/A588M for atmospheric corrosion resistance, the requirements in Section 6.1.1(c) are applicable.

6.1.2. During fabrication, up to the point of assembling members, each piece of material that is ordered to special material requirements shall carry a Fabricator's identification mark or an original supplier's identification mark. The Fabricator's identification mark shall be in accordance with the Fabricator's established material identification system, which shall be on record and available prior to the start of fabrication for the information of the Owner's Designated Representative for Construction, the building-code authority and the Inspector.

6.1.3. Members that are made of material that is ordered to special material requirements shall not be given the same assembling or erection mark as members made of other material, even if they are of identical dimensions and detail.

6.2. Preparation of Material

6.2.1. The thermal cutting of Structural Steel by hand-guided or mechanically guided means is permitted.

6.2.2. Surfaces that are specified as "finished" in the Contract Documents shall have a roughness height value measured in accordance with ANSI/ASME B46.1 that is equal to or less than 500. The use of any fabricating technique that produces such a finish is permitted.

Commentary:
Most cutting processes, including friction sawing and cold sawing, and milling processes meet a surface roughness limitation of 500 per ANSI/ASME B46.1.

6.3. Fitting and Fastening

6.3.1. Projecting elements of Connection materials need not be straightened in the connecting plane, subject to the limitations in the AISC Specification.

6.3.2. Backing bars and runoff tabs shall be used in accordance with AWS D1.1 as required to produce sound welds. The Fabricator or Erector need not remove backing bars or runoff tabs unless such removal is specified in the Contract Documents. When the removal of backing bars is specified in the Contract Documents, such removal shall meet the requirements in AWS D1.1. When the removal of runoff tabs is specified in the Contract Documents, hand flame-cutting close to the edge of the finished member with no further finishing is permitted, unless other finishing is specified in the Contract Documents.

Commentary:
In most cases, the treatment of backing bars and runoff tabs is left to the discretion of the Owner's Designated Representative for Design. In some cases, treatment beyond the basic cases described in this Section may be required. As one example, special treatment is required for backing bars and runoff tabs in beam-to-column moment Connections when the requirements in the AISC Seismic Provisions must be met. In all cases, the Owner's Designated Representative for Design should specify the required treatments in the Contract Documents.

6.3.3. Unless otherwise noted in the Shop Drawings, high-strength bolts for shop-attached Connection material shall be installed in the shop in accordance with the requirements in the AISC Specification.

6.4. Fabrication Tolerances

The tolerances on Structural Steel fabrication shall be in accordance with the requirements in Section 6.4.1 through 6.4.6.

Commentary:
Fabrication tolerances are stipulated in several specifications and codes, each applicable to a specialized area of construction. Basic fabrication tolerances are stipulated in this Section. For Architecturally Exposed Structural Steel, see Section 10. Other specifications and codes are also commonly incorporated by reference in the Contract Documents, such as the AISC Specification, the RCSC Specification, AWS D1.1 and the AASHTO Specification.

6.4.1. For members that have both ends finished (see Section 6.2.2) for contact bearing, the variation in the overall length shall be equal to or less than 1/32 in. [1 mm]. For other members that frame to other Structural Steel elements, the variation in the detailed length shall be as follows:

(a) For members that are equal to or less than 30 ft [9 000 mm] in length, the variation shall be equal to or less than 1/16 in. [2 mm].
(b) For members that are greater than 30 ft [9 000 mm] in length, the variation shall be equal to or less than 1/8 in. [3 mm].

6.4.2. For straight structural members other than compression members, whether of a single Standard Structural Shape or built-up, the variation in straightness shall be equal to or less than that specified for wide-flange shapes in ASTM A6/A6M, except when a smaller variation in straightness is specified in the Contract Documents. For straight compression members, whether of a Standard

Structural Shape or built-up, the variation in straightness shall be equal to or less than 1/1000 of the axial length between points that are to be laterally supported. For curved structural members, the variation from the theoretical curvature shall be equal to or less than the variation in sweep that is specified for an equivalent straight member of the same straight length in ASTM A6/A6M.

In all cases, completed members shall be free of twists, bends and open joints. Sharp kinks or bends shall be cause for rejection.

6.4.3. For beams and trusses that are detailed without specified camber, the member shall be fabricated so that, after erection, any incidental camber due to rolling or shop fabrication is upward.

6.4.4. For beams that are specified in the Contract Documents with camber, beams received by the Fabricator with 75% of the specified camber shall require no further cambering. Otherwise, the variation in camber shall be as follows:

(a) For beams that are equal to or less than 50 ft [15 000 mm] in length, the variation shall be equal to or less than minus zero / plus 1/2 in. [13 mm].
(b) For beams that are greater than 50 ft [15 000 mm] in length, the variation shall be equal to or less than minus zero / plus 1/2 in. plus 1/8 in. for each 10 ft or fraction thereof [13 mm plus 3 mm for each 3 000 mm or fraction thereof] in excess of 50 ft [15 000 mm] in length.

For the purpose of inspection, camber shall be measured in the Fabricator's shop in the unstressed condition.

Commentary:
There is no known way to inspect beam camber after the beam is received in the field because of factors that include:

(a) The release of stresses in members over time and in varying applications;
(b) The effects of the dead weight of the member;
(c) The restraint caused by the end Connections in the erected state; and,
(d) The effects of additional dead load that may ultimately be intended to be applied, if any.

Therefore, inspection of the Fabricator's work on beam camber must be done in the fabrication shop in the unstressed condition.

6.4.5. For fabricated trusses that are specified in the Contract Documents with camber, the variation in camber at each specified camber point shall be equal to or less than plus or minus 1/800 of the distance to that point from the nearest point of support. For the purpose of inspection, camber shall be measured in the Fabricator's shop in the unstressed condition. For fabricated trusses that are specified in the Contract Documents without indication of camber, the foregoing requirements shall be applied at each panel point of the truss with a zero camber ordinate.

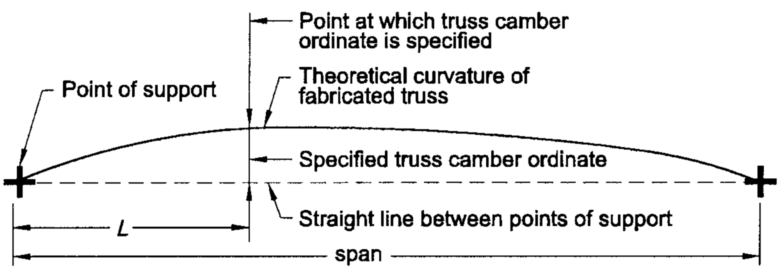
Commentary:
There is no known way to inspect truss camber after the truss is received in the field because of factors that include:

(a) The effects of the dead weight of the member;
(b) The restraint caused by the truss Connections in the erected state; and,
(c) The effects of additional dead load that may ultimately be intended to be applied, if any.

Therefore, inspection of the Fabricator's work on truss camber must be done in the fabrication shop in the unstressed condition. See Figure C–6.1.

6.4.6. When permissible variations in the depths of beams and girders result in abrupt changes in depth at splices, such deviations shall be accounted for as follows:

(a) For splices with bolted joints, the variations in depth shall be taken up with filler plates; and,
(b) For splices with welded joints, the weld profile shall be adjusted to conform to the variations in depth, the required cross-section of weld shall be provided and the slope of the weld surface shall meet the requirements in AWS D1.1.



Taking *L* as the distance from the point at which truss camber is specified to the closer point of support, in. [mm], the tolerance on truss camber at that point is calculated as *L*/800. *L* must be equal to or less than one-half the span.

Figure C-6.1. Illustration of the tolerance on camber for fabricated trusses with specified camber.

6.5. Shop Cleaning and Painting (see also Section 3.1.6)

Structural Steel that does not require shop paint shall be cleaned of oil and grease with solvent cleaners, and of dirt and other foreign material by sweeping with a fiber brush or other suitable means. For Structural Steel that is required to be shop painted, the requirements in Sections 6.5.1 through 6.5.4 shall apply.

Commentary:

Extended exposure of unpainted Structural Steel that has been cleaned for the subsequent application of fire protection materials can be detrimental to the fabricated product. Most levels of cleaning require the removal of all loose mill scale, but permit some amount of tightly adhering mill scale. When a piece of Structural Steel that has been cleaned to an acceptable level is left exposed to a normal environment, moisture can penetrate behind the scale, and some “lifting” of the scale by the oxidation process is to be expected. Cleanup of “lifted” mill scale is not the responsibility of the Fabricator, but is to be assigned by contract requirement to an appropriate contractor.

Section 6.5.4 of this Code is not applicable to weathering steel, for which special cleaning specifications are always required in the Contract Documents.

6.5.1. The Fabricator is not responsible for deterioration of the shop coat that may result from exposure to ordinary atmospheric conditions or corrosive conditions that are more severe than ordinary atmospheric conditions.

Commentary:

The shop coat of paint is the prime coat of the protective system. It is intended as protection for only a short period of exposure in ordinary atmospheric conditions, and is considered a temporary and provisional coating.

6.5.2. Unless otherwise specified in the Contract Documents, the Fabricator shall, as a minimum, hand clean the Structural Steel of loose rust, loose mill scale, dirt and other foreign matter, prior to painting, by means of wire brushing or by other methods elected by the Fabricator, to meet the requirements of SSPC-SP2. If the Fabricator’s workmanship on surface preparation is to be inspected by the Inspector, such inspection shall be performed in a timely manner prior to the application of the shop coat.

Commentary:

The selection of a paint system is a design decision involving many factors including:

(a) The Owner’s preference;

(b) The service life of the structure;
(c) The severity of environmental exposure;
(d) The cost of both initial application and future renewals; and,
(e) The compatibility of the various components that comprise the paint system (surface preparation, shop coat and subsequent coats).

Because the inspection of shop painting must be concerned with workmanship at each stage of the operation, the Fabricator provides notice of the schedule of operations and affords the Inspector access to the work site. Inspection must then be coordinated with that schedule so as to avoid delay of the scheduled operations.

Acceptance of the prepared surface must be made prior to the application of the shop coat because the degree of surface preparation cannot be readily verified after painting. Time delay between surface preparation and the application of the shop coat can result in unacceptable deterioration of a properly prepared surface, necessitating a repetition of surface preparation. This is especially true with blast-cleaned surfaces. Therefore, to avoid potential deterioration of the surface, it is assumed that surface preparation is accepted unless it is inspected and rejected prior to the scheduled application of the shop coat.

The shop coat in any paint system is designed to maximize the wetting and adherence characteristics of the paint, usually at the expense of its weathering capabilities. Deterioration of the shop coat normally begins immediately after exposure to the elements and worsens as the duration of exposure is extended. Consequently, extended exposure of the shop coat will likely lead to its deterioration and may necessitate repair, possibly including the repetition of surface preparation and shop coat application in limited areas. With the introduction of high-performance paint systems, avoiding delay in the application of the shop coat has become more critical. High-performance paint systems generally require a greater degree of surface preparation, as well as early application of weathering protection for the shop coat.

Since the Fabricator does not control the selection of the paint system, the compatibility of the various components of the total paint system, or the length of exposure of the shop coat, the Fabricator cannot guarantee the performance of the shop coat or any other part of the system. Instead, the Fabricator is responsible only for accomplishing the specified surface preparation and for applying the shop coat (or coats) in accordance with the Contract Documents.

This Section stipulates that the Structural Steel is to be cleaned to meet the requirements in SSPC-SP2. This stipulation is not intended to represent an exclusive cleaning level, but rather the level of surface preparation that will be furnished unless otherwise specified in the Contract Documents if the Structural Steel is to be painted.

Further information regarding shop painting is available in *A Guide to Shop Painting of Structural Steel*, published jointly by SSPC and AISC.

6.5.3. Unless otherwise specified in the Contract Documents, paint shall be applied by brushing, spraying, rolling, flow coating, dipping or other suitable means, at the election of the Fabricator. When the term "shop coat", "shop paint" or other equivalent term is used with no paint system specified, the Fabricator's standard shop paint shall be applied to a minimum dry-film thickness of one mil [25 μm].

6.5.4. Touch-up of abrasions caused by handling after painting shall be the responsibility of the contractor that performs touch-up in the field or field painting.

Commentary:
Touch-up in the field and field painting are not normally part of the Fabricator's or the Erector's contract.

6.6. Marking and Shipping of Materials

6.6.1. Unless otherwise specified in the Contract Documents, erection marks shall be applied to the Structural Steel members by painting or other suitable means.

6.6.2. Bolt assemblies and loose bolts, nuts and washers shall be shipped in separate closed containers according to length and diameter, as applicable. Pins and other small parts and packages of bolts, nuts and washers shall be shipped in boxes, crates, kegs or barrels. A list and description of the material shall appear on the outside of each closed container.

Commentary:
In most cases bolts, nuts and other components in a fastener assembly can be shipped loose in separate containers. However, ASTM F1852/F1852M twist-off-type tension-control bolt assemblies and galvanized ASTM A325, A325M and F1852/F1852M bolt assemblies must be assembled and shipped in the same container according to length and diameter.

6.7. Delivery of Materials

6.7.1. Fabricated Structural Steel shall be delivered in a sequence that will permit efficient and economical fabrication and erection, and that is consistent with requirements in the Contract Documents. If the Owner or Owner's Designated Representative for Construction wishes to prescribe or control the sequence of delivery of materials, that entity shall specify the required sequence in the Contract Documents. If the Owner's Designated Representative for Construction contracts separately for delivery and for erection, the Owner's

Designated Representative for Construction shall coordinate planning between contractors.

6.7.2. Anchor Rods, washers, nuts and other anchorage or grillage materials that are to be built into concrete or masonry shall be shipped so that they will be available when needed. The Owner's Designated Representative for Construction shall allow the Fabricator sufficient time to fabricate and ship such materials before they are needed.

6.7.3. If any shortage is claimed relative to the quantities of materials that are shown in the shipping statements, the Owner's Designated Representative for Construction or the Erector shall promptly notify the Fabricator so that the claim can be investigated.

Commentary:
The quantities of material that are shown in the shipping statement are customarily accepted as correct by the Owner's Designated Representative for Construction, the Fabricator and the Erector.

6.7.4. Unless otherwise specified in the Contract Documents, and subject to the approved Shop and Erection Drawings, the Fabricator shall limit the number of field splices to that consistent with minimum project cost.

Commentary:
This Section recognizes that the size and weight of Structural Steel assemblies may be limited by shop capabilities, the permissible weight and clearance dimensions of available transportation or job-site conditions.

6.7.5. If material arrives at its destination in damaged condition, the receiving entity shall promptly notify the Fabricator and carrier prior to unloading the material, or promptly upon discovery prior to erection.

SECTION 7. ERECTION

7.1. Method of Erection

Fabricated Structural Steel shall be erected using methods and a sequence that will permit efficient and economical performance of erection, and that is consistent with the requirements in the Contract Documents. If the Owner or Owner's Designated Representative for Construction wishes to prescribe or control the method and/or sequence of erection, or specifies that certain members cannot be erected in their normal sequence, that entity shall specify the required method and sequence in the Contract Documents. If the Owner's Designated Representative for Construction contracts separately for fabrication services and for erection services, the Owner's Designated Representative for Construction shall coordinate planning between contractors.

Commentary:
Design modifications are sometimes requested by the Erector to allow or facilitate the erection of the Structural Steel frame. When this is the case, the Erector should notify the Fabricator prior to the preparation of Shop and Erection Drawings so that the Fabricator may refer the Erector's request to the Owner's Designated Representatives for Design and Construction for resolution.

7.2. Job-Site Conditions

The Owner's Designated Representative for Construction shall provide and maintain the following for the Fabricator and the Erector:

(a) Adequate access roads into and through the job site for the safe delivery and movement of the material to be erected and of derricks, cranes, trucks and other necessary equipment under their own power;
(b) A firm, properly graded, drained, convenient and adequate space at the job site for the operation of the Erector's equipment, free from overhead obstructions, such as power lines, telephone lines or similar conditions; and,
(c) Adequate storage space, when the structure does not occupy the full available job site, to enable the Fabricator and the Erector to operate at maximum practical speed.

Otherwise, the Owner's Designated Representative for Construction shall inform the Fabricator and the Erector of the actual job-site conditions and/or special delivery requirements prior to bidding.

7.3. Foundations, Piers and Abutments

The accurate location, strength and suitability of, and access to, all foundations, piers and abutments shall be the responsibility of the Owner's Designated Representative for Construction.

7.4. Lines and Bench Marks

The Owner's Designated Representative for Construction shall be responsible for the accurate location of lines and benchmarks at the job site and shall furnish the Erector with a plan that contains all such information. The Owner's Designated Representative for Construction shall establish offset lines and reference elevations at each level for the Erector's use in the positioning of Adjustable Items (see Section 7.13.1.3), if any.

7.5. Installation of Anchor Rods, Foundation Bolts and Other Embedded Items

7.5.1. Anchor Rods, foundation bolts and other embedded items shall be set by the Owner's Designated Representative for Construction in accordance with Embedment Drawings that have been approved by the Owner's Designated Representatives for Design and Construction. The variation in location of these items from the dimensions shown in the Embedment Drawings shall be as follows:

(a) The variation in dimension between the centers of any two Anchor Rods within an Anchor-Rod Group shall be equal to or less than 1/8 in. [3 mm].
(b) The variation in dimension between the centers of adjacent Anchor-Rod Groups shall be equal to or less than 1/4 in. [6 mm].
(c) The variation in elevation of the tops of Anchor Rods shall be equal to or less than plus or minus 1/2 in. [13 mm].
(d) The accumulated variation in dimension between centers of Anchor-Rod Groups along the Column Line through multiple Anchor-Rod Groups shall be equal to or less than 1/4 in. per 100 ft [2 mm per 10 000 mm], but not to exceed a total of 1 in. [25 mm].
(e) The variation in dimension from the center of any Anchor-Rod Group to the Column Line through that group shall be equal to or less than 1/4 in. [6 mm].

The tolerances that are specified in (b), (c) and (d) shall apply to offset dimensions shown in the structural Design Drawings, measured parallel and perpendicular to the nearest Column Line, for individual columns that are shown in the structural Design Drawings as offset from Column Lines.

Commentary:
The tolerances established in this Section have been selected for compatibility with the holes sizes that are recommended for base plates in the AISC Manual of Steel Construction. If special conditions require more restrictive tolerances, the contractor responsible for setting the Anchor Rods should be so informed in the Contract Documents. When the Anchor Rods are set in sleeves, the

adjustment provided may be used to satisfy the required Anchor-Rod setting tolerances.

7.5.2. Unless otherwise specified in the Contract Documents, Anchor Rods shall be set with their longitudinal axis perpendicular to the theoretical bearing surface.

7.5.3. Embedded items and Connection materials that are part of the work of other trades, but that will receive Structural Steel, shall be located and set by the Owner's Designated Representative for Construction in accordance with an approved Embedment Drawing. The variation in location of these items shall be limited to a magnitude that is consistent with the tolerances that are specified in Section 7.13 for the erection of the Structural Steel.

7.5.4. All work that is performed by the Owner's Designated Representative for Construction shall be completed so as not to delay or interfere with the work of the Fabricator and the Erector. The Owner's Designated Representative for Construction shall conduct a survey of the as-built locations of Anchor Rods, foundation bolts and other embedded items, and shall verify that all items covered in Section 7.5 meet the corresponding tolerances. When corrective action is necessary, the Owner's Designated Representative for Construction shall obtain the guidance and approval of the Owner's Designated Representative for Design.

Commentary:
Few Fabricators or Erectors have the capability to provide this survey. Under standard practice, it is the responsibility of others.

7.6. Installation of Bearing Devices

All leveling plates, leveling nuts and washers and loose base and bearing plates that can be handled without a derrick or crane are set to line and grade by the Owner's Designated Representative for Construction. Loose base and bearing plates that require handling with a derrick or crane shall be set by the Erector to lines and grades established by the Owner's Designated Representative for Construction. The Fabricator shall clearly scribe loose base and bearing plates with lines or other suitable marks to facilitate proper alignment.

Promptly after the setting of Bearing Devices, the Owner's Designated Representative for Construction shall check them for line and grade. The variation in elevation relative to the established grade for all Bearing Devices shall be equal to or less than plus or minus 1/8 in. [3 mm]. The final location of Bearing Devices shall be the responsibility of the Owner's Designated Representative for Construction.

Commentary:
The 1/8 in. [3 mm] tolerance on elevation of Bearing Devices relative to established grades is provided to permit some variation in setting Bearing Devices, and to account for the accuracy that is attainable with standard surveying instruments. The use of leveling plates larger than 22 in. by 22 in. [550 mm by 550 mm] is discouraged and grouting is recommended with larger sizes. For the purposes of erection stability, the use of leveling nuts and washers is discouraged when base plates have less than four Anchor Rods.

7.7. Grouting

Grouting shall be the responsibility of the Owner's Designated Representative for Construction. Leveling plates and loose base and bearing plates shall be promptly grouted after they are set and checked for line and grade. Columns with attached base plates, beams with attached bearing plates and other similar members with attached Bearing Devices that are temporarily supported on leveling nuts and washers, shims or other similar leveling devices, shall be promptly grouted after the Structural Steel frame or portion thereof has been plumbed.

Commentary:
In the majority of structures the vertical load from the column bases is transmitted to the foundations through structural grout. In general, there are three methods by which support is provided for column bases during erection:

(a) Pre-grouted leveling plates or loose base plates;
(b) Shims; and,
(c) Leveling nuts and washers on the Anchor Rods beneath the column base.

Standard practice provides that loose base plates and leveling plates are to be grouted as they are set. Bearing Devices that are set on shims or leveling nuts are grouted after plumbing, which means that the weight of the erected Structural Steel frame is supported on the shims or washers, nuts and Anchor Rods. The Erector must take care to ensure that the load that is transmitted in this temporary condition does not exceed the strength of the shims or washers, nuts and Anchor Rods. These considerations are presented in greater detail in AISC Design Guides No. 1 and 10.

7.8. Field Connection Material

7.8.1. The Fabricator shall provide field Connection details that are consistent with the requirements in the Contract Documents and that will, in the Fabricator's opinion, result in economical fabrication and erection.

7.8.2. When the Fabricator is responsible for erecting the Structural Steel, the Fabricator shall furnish all materials that are required for both temporary and permanent Connection of the component parts of the Structural Steel frame.

7.8.3. When the erection of the Structural Steel is not performed by the Fabricator, the Fabricator shall furnish the following field Connection material:

(a) Bolts, nuts and washers of the required grade, type and size and in sufficient quantity for all Structural Steel-to-Structural Steel field Connections that are to be permanently bolted, including an extra 2 percent of each bolt size (diameter and length);
(b) Shims that are shown as necessary for make-up of permanent Structural Steel-to-Structural Steel Connections; and,
(c) Backing bars and run-off tabs that are required for field welding.

7.8.4. The Erector shall furnish all welding electrodes, fit-up bolts and drift pins used for the erection of the Structural Steel.

Commentary:
See the commentary for Section 2.2.

7.9. Loose Material

Unless otherwise specified in the Contract Documents, loose Structural Steel items that are not connected to the Structural Steel frame shall be set by the Owner's Designated Representative for Construction without assistance from the Erector.

7.10. Temporary Support of Structural Steel Frames

7.10.1. The Owner's Designated Representative for Design shall identify the following in the Contract Documents:

(a) The lateral-load-resisting system and connecting diaphragm elements that provide for lateral strength and stability in the completed structure; and,
(b) Any special erection conditions or other considerations that are required by the design concept, such as the use of shores, jacks or loads that must be adjusted as erection progresses to set or maintain camber, position within specified tolerances or prestress.

Commentary:
The intent of Section 7.10.1 of the Code is to alert the Owners Designated Representative for Construction and the Erector of the means for lateral load resistance in the completed structure so that appropriate planning can occur for

construction of the building. Examples of a description of the lateral load resisting system as required by 7.10.1(a) are shown below.

Example 1 is an all-steel building with a composite metal deck and concrete floor system. All lateral load resistance is provided by welded moment frames in each orthogonal building direction. One suitable description of this lateral load resisting system is:

All lateral load resistance and stability of the building in the completed structure is provided by moment frames with welded beam to column connections framed in each orthogonal direction (see plan sheets for locations). The composite metal deck and concrete floors serve as horizontal diaphragms that distribute the lateral wind and seismic forces horizontally to the vertical moment frames. The vertical moment frames carry the applied lateral loads to the building foundation.

Example 2 is a steel-framed building with a composite metal deck and concrete floor system. All beam-to-column connections are simple connections and all lateral load resistance is provided by reinforced concrete shear walls in the building core and in the stair wells. One suitable description of this lateral load resisting system is:

All lateral load resistance and stability of the building in the completed structure is provided exclusively by cast-in-place reinforced concrete shear walls in the building core and stair wells (see plan sheets for locations). These walls provide all lateral load resistance in each orthogonal building direction. The composite metal deck and concrete floors serve as horizontal diaphragms that distribute the lateral wind and seismic forces horizontally to the concrete shear walls. The concrete shear walls carry the applied lateral loads to the building foundation.

See also Commentary Section 7.10.3.

7.10.2. The Owner's Designated Representative for Construction shall indicate to the Erector prior to bidding, the installation schedule for non-Structural Steel elements of the lateral-load-resisting system and connecting diaphragm elements identified by the Owner's Designated Representative for Design in the Contract Documents.

Commentary:
See Commentary Section 7.10.3.

7.10.3. Based upon the information provided in accordance with Sections 7.10.1 and 7.10.2, the Erector shall determine, furnish and install all temporary supports, such as temporary guys, beams, falsework, cribbing or other elements required for the erection operation. These temporary supports shall be sufficient to secure the bare Structural Steel framing or any portion thereof against loads that are likely to be encountered during erection, including those due to wind and those that result from erection operations.

The Erector need not consider loads during erection that result from the performance of work by, or the acts of, others, except as specifically identified by the Owner's Designated Representatives for Design and Construction, nor those that are unpredictable, such as loads due to hurricane, tornado, earthquake, explosion or collision.

Temporary supports that are required during or after the erection of the Structural Steel frame for the support of loads caused by non-Structural Steel elements, including cladding, interior partitions and other such elements that will induce or transmit loads to the Structural Steel frame during or after erection, shall be the responsibility of others.

Commentary:
Many Structural Steel frames have lateral-load-resisting systems that are activated during the erection process. Such lateral-load-resisting systems may consist of welded moment frames, braced frames or, in some instances, columns that cantilever from fixed-base foundations. Such frames are normally braced with temporary guys that, together with the steel deck floor and roof diaphragms, or other diaphragm bracing that may be included as part of the design, provide stability during the erection process. The guy cables are also commonly used to plumb the Structural Steel frame. The Erector normally furnishes and installs the required temporary supports and bracing to secure the bare Structural Steel frame, or portion thereof, during the erection process. When Erection Bracing Drawings are required in the Contract Documents, those drawings show this information.

If the Owner's Designated Representative for Construction determines that steel decking is not installed by the Erector, temporary diaphragm bracing may be required if a horizontal diaphragm is not available to distribute loads to the vertical and lateral load resisting system. If the steel deck will not be available as a diaphragm during Structural Steel erection, the Owner's Designated Representative for Construction must communicate this condition to the Erector prior to bidding. If such diaphragm bracing is required, it must be furnished and installed by the Erector.

Sometimes structural systems that are employed by the Owner's Designated Representative for Design rely upon other elements besides the Structural Steel frame for lateral-load resistance. For instance, concrete or masonry shear walls or precast spandrels may be used to provide resistance to vertical and lateral loads in the completed structure. Because these situations may not be obvious to the contractor or the Erector, it is required in this Code that the Owner's Designated Representative for Design identify such situations in the Contract Documents. Similarly, if a structure is designed so that special erection techniques are required, such as jacking to impose certain loads or position during erection, it is required in this Code that such requirements be specifically identified in the Contract Documents.

In some instances, the Owner's Designated Representative for Design may elect to show erection bracing in the Design Drawings. When this is the case, the Owner's Designated Representative for Design should then confirm that the bracing requirements were understood by review and approval of the Erection Drawings during the submittal process.

Sometimes during construction of a building, collateral building elements, such as exterior cladding, may be required to be installed on the bare Structural Steel frame prior to completion of the lateral-load-resisting system. These elements may increase the potential for lateral loads on the temporary supports. Such temporary supports may also be required to be left in place after the Structural Steel frame has been erected. Special provisions should be made by the Owner's Designated Representative for Construction for these conditions.

7.10.4. All temporary supports that are required for the erection operation and furnished and installed by the Erector shall remain the property of the Erector and shall not be modified, moved or removed without the consent of the Erector. Temporary supports provided by the Erector shall remain in place until the portion of the Structural Steel frame that they brace is complete and the lateral-load-resisting system and connecting diaphragm elements identified by the Owner's Designated Representative for Design in accordance with Section 7.10.1 are installed. Temporary supports that are required to be left in place after the completion of Structural Steel erection shall be removed when no longer needed by the Owner's Designated Representative for Construction and returned to the Erector in good condition.

7.11. Safety Protection

7.11.1. The Erector shall provide floor coverings, handrails, walkways and other safety protection for the Erector's personnel as required by law and the applicable safety regulations. Unless otherwise specified in the Contract Documents, the Erector is permitted to remove such safety protection from areas where the erection operations are completed.

7.11.2. When safety protection provided by the Erector is left in an area for the use of other trades after the Structural Steel erection activity is completed, the Owner's Designated Representative for Construction shall:

(a) Accept responsibility for and maintain this protection;
(b) Indemnify the Fabricator and the Erector from damages that may be incurred from the use of this protection by other trades;
(c) Ensure that this protection is adequate for use by other affected trades;

(d) Ensure that this protection complies with applicable safety regulations when being used by other trades; and,
(e) Remove this protection when it is no longer required and return it to the Erector in the same condition as it was received.

7.11.3. Safety protection for other trades that are not under the direct employment of the Erector shall be the responsibility of the Owner's Designated Representative for Construction.

7.11.4. When permanent steel decking is used for protective flooring and is installed by the Owner's Designated Representative for Construction, all such work shall be scheduled and performed in a timely manner so as not to interfere with or delay the work of the Fabricator or the Erector. The sequence of installation that is used shall meet all safety regulations.

7.11.5. Unless the interaction and safety of activities of others, such as construction by others or the storage of materials that belong to others, are coordinated with the work of the Erector by the Owner's Designated Representative for Construction, such activities shall not be permitted until the erection of the Structural Steel frame or portion thereof is completed by the Erector and accepted by the Owner's Designated Representative for Construction.

7.12. Structural Steel Frame Tolerances

The accumulation of the mill tolerances and fabrication tolerances shall not cause the erection tolerances to be exceeded.

Commentary:
In previous editions of this Code, it was stated that "…variations are deemed to be within the limits of good practice when they do not exceed the cumulative effect of rolling tolerances, fabricating tolerances and erection tolerances." It is recognized in the current provision in this Section that accumulations of mill tolerances and fabrication tolerances generally occur between the locations at which erection tolerances are applied, and not at the same locations.

7.13. Erection Tolerances

Erection tolerances shall be defined relative to member working points and working lines, which shall be defined as follows:

(a) For members other than horizontal members, the member work point shall be the actual center of the member at each end of the shipping piece.
(b) For horizontal members, the working point shall be the actual centerline of the top flange or top surface at each end.
(c) The member working line shall be the straight line that connects the member working points.

The substitution of other working points is permitted for ease of reference, provided they are based upon the above definitions.

The tolerances on Structural Steel erection shall be in accordance with the requirements in Sections 7.13.1 through 7.13.3.

Commentary:
The erection tolerances defined in this Section have been developed through long-standing usage as practical criteria for the erection of Structural Steel. Erection tolerances were first defined in the 1924 edition of this Code in Section 7(f), “Plumbing Up.” With the changes that took place in the types and use of materials in building construction after World War II, and the increasing demand by Architects and Owners for more specific tolerances, AISC adopted new standards for erection tolerances in Section 7(h) of the March 15, 1959 edition of this Code. Experience has proven that those tolerances can be economically obtained.

Differential column shortening may be a consideration in design and construction. In some cases, it may occur due to variability in the accumulation of dead load among different columns (see Figure C–7.1). In other cases, it may be characteristic of the structural system that is employed in the design. Consideration of the effects of differential column shortening may be very important, such as when the slab thickness is reduced, when electrical and other similar fittings mounted on the Structural Steel are intended to be flush with the finished floor and when there is little clearance between bottoms of beams and the tops of door frames or ductwork.

Expansion and contraction in a Structural Steel frame may also be a consideration in the design and construction. Steel will expand or contract approximately 1/8 in. per 100 ft for each change of 15°F [2 mm per 10 000 mm for each change of 15°C] in temperature. This change in length can be assumed to act about the center of rigidity. When anchored to their foundations, end columns will be plumb only when the steel is at normal temperature (see Figure C–7.2). It is therefore necessary to correct field measurements of offsets to the structure from established baselines for the expansion or contraction of the exposed Structural Steel frame. For example, a 200-ft-long [60 000-m-long] building that is plumbed up at 100°F [38°C] should have working points at the tops of the end columns positioned 1/2 in. [14 mm] further apart than the working points at the corresponding bases in order for the columns to be plumb at 70°F [21°C]. Differential temperature effects on column length should also be taken into account in plumbing surveys when tall Structural Steel frames are subjected to sun exposure on one side.

The alignment of lintels, spandrels, wall supports and similar members that are used to connect other building construction units to the Structural Steel frame should have an adjustment of sufficient magnitude to allow for the accumulation of mill tolerances and fabrication tolerances, as well as the erection tolerances. See Figure C–7.3.

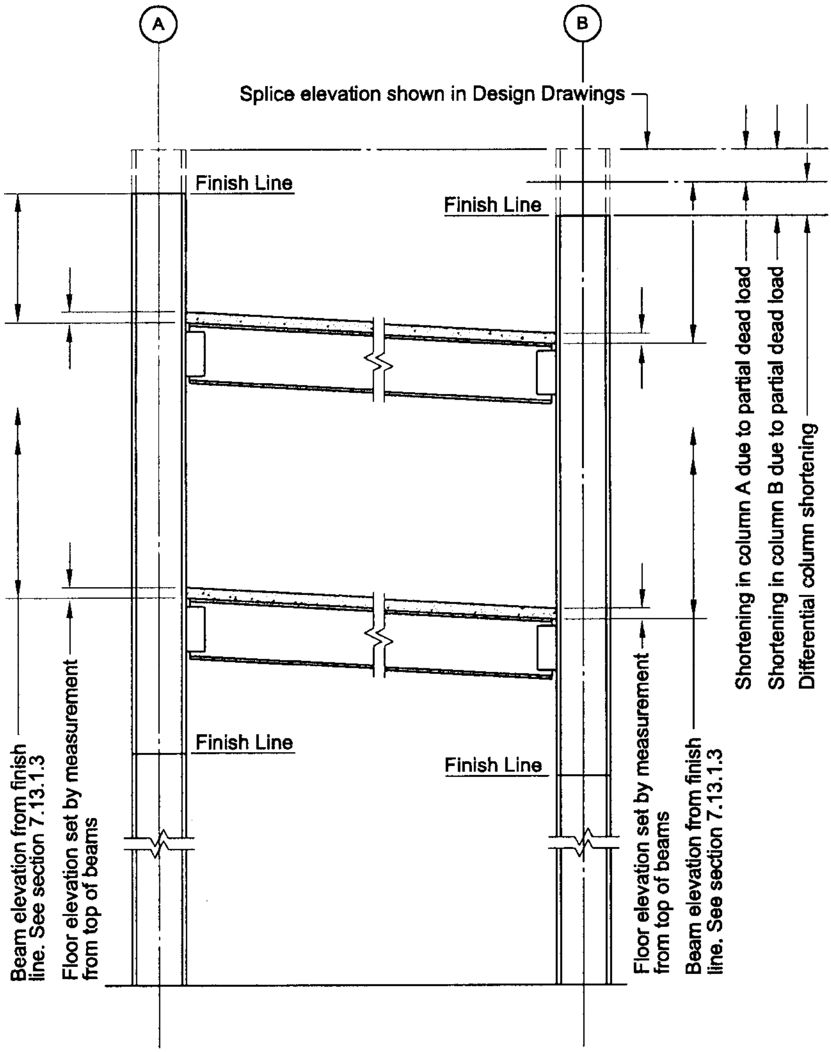


Figure C-7.1. Effects of differential column shortening.

When plumbing columns, apply a temperature adjustment at a rate of 1/8 in. per 100 ft. for each change of 15° F [2 mm per 10 000 mm for each change of 15° C] between the temperature at the time of erection and the working temperature.

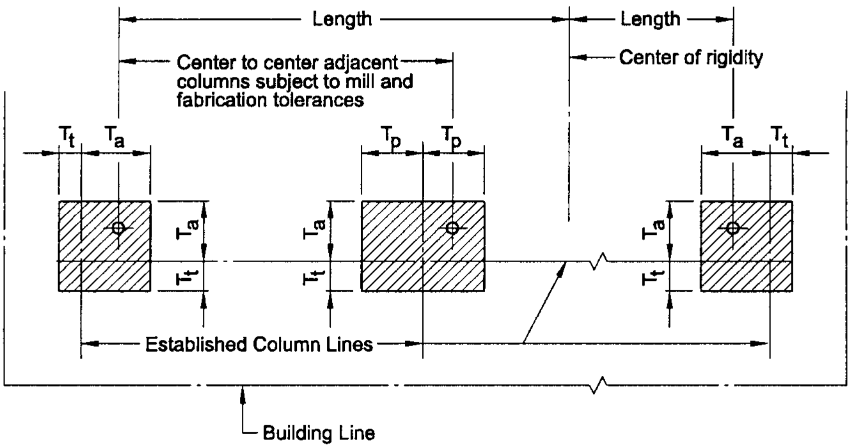


Figure C-7.2. Tolerances in plan location of column.

7.13.1. The tolerances on position and alignment of member working points and working lines shall be as described in Sections 7.13.1.1 through 7.13.1.3.

7.13.1.1. For an individual column shipping piece, the angular variation of the working line from a plumb line shall be equal to or less than 1/500 of the distance between working points, subject to the following additional limitations:

(a) For an individual column shipping piece that is adjacent to an elevator shaft, the displacement of member working points shall be equal to or less than 1 in. [25 mm] from the Established Column Line in the first 20 stories. Above this level, an increase in the displacement of 1/32 in. [1 mm] is permitted for each additional story up to a maximum displacement of 2 in. [50 mm] from the Established Column Line.

(b) For an exterior individual column shipping piece, the displacement of member working points from the Established Column Line in the first 20 stories shall be equal to or less than 1 in. [25 mm] toward and 2 in. [50 mm] away from the building line. Above this level, an increase in the displacement of 1/16 in. [2 mm] is permitted for each additional story up to a maximum displacement of 2 in. [50 mm] toward and 3 in. [75 mm] away from the building line.

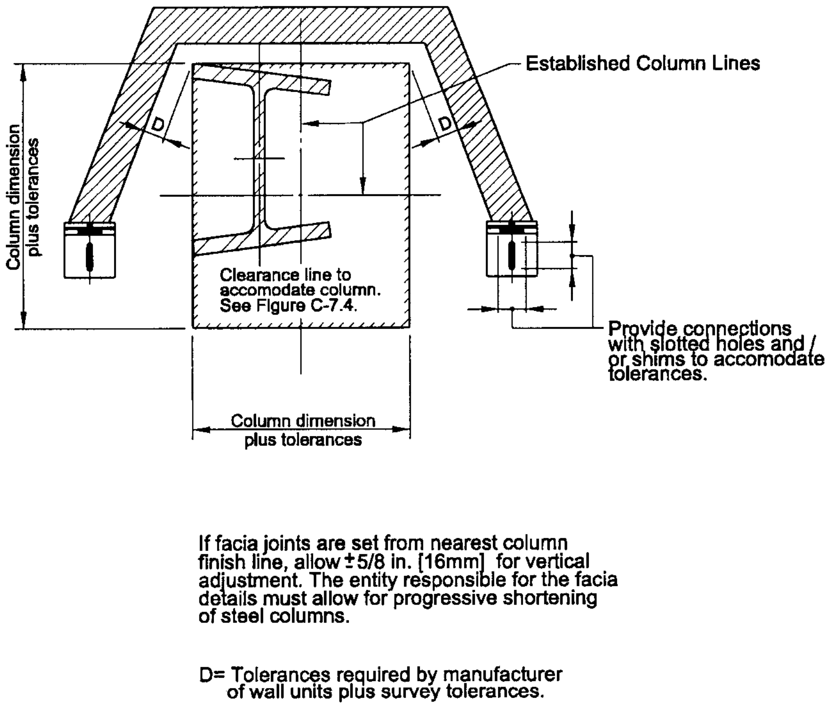


Figure C-7.3.Clearance required to accommodate fascia.

Commentary:
The limitations that are described in this Section and illustrated in Figures C–7.4 and C–7.5 make it possible to maintain built-in-place or prefabricated facades in a true vertical plane up to the 20th story, if Connections that provide for 3 in. [75 mm] of adjustment are used. Above the 20th story, the facade may be maintained within 1/16 in. [2 mm] per story with a maximum total deviation of 1 in. [25 mm] from a true vertical plane, if Connections that provide for 3 in. [75 mm] of adjustment are used. Connections that permit adjustments of plus 2 in. [50 mm] to minus 3 in. [75 mm] (5 in. [125 mm] total) will be necessary in cases where it is desired to construct the facade to a true vertical plane above the 20th story.

(c) For an exterior individual column shipping piece, the member working points at any splice level for multi-Tier buildings and at the tops of columns for single-Tier buildings shall fall within a horizontal envelope, parallel to the building line, that is equal to or less than 1 1/2 in. [38 mm] wide for buildings up to 300 ft [90 000 mm] in length. An increase in the width of this horizontal envelope of 1/2 in. [13 mm] is permitted for each additional 100 ft [30 000 m] in length up to a maximum width of 3 in. [75 mm].

Commentary:
This Section limits the position of exterior column working points at any given splice elevation to a narrow horizontal envelope parallel to the building line (see Figure C–7.6). This envelope is limited to a width of 1 1/2 in. [38 mm], normal to the building line, in up to 300 ft [90 000 mm] of building length. The horizontal location of this envelope is not necessarily directly above or below the corresponding envelope at the adjacent splice elevations, but should be within the limitation of the 1 in 500 plumbness tolerance specified for the controlling columns (see Figure C–7.5).

(d) For an exterior column shipping piece, the displacement of member working points from the Established Column Line, parallel to the building line, shall be equal to or less than 2 in. [50 mm] in the first 20 stories. Above this level, an increase in the displacement of 1/16 in. [2 mm] is permitted for each additional story up to a maximum displacement of 3 in. [75 mm] parallel to the building line.

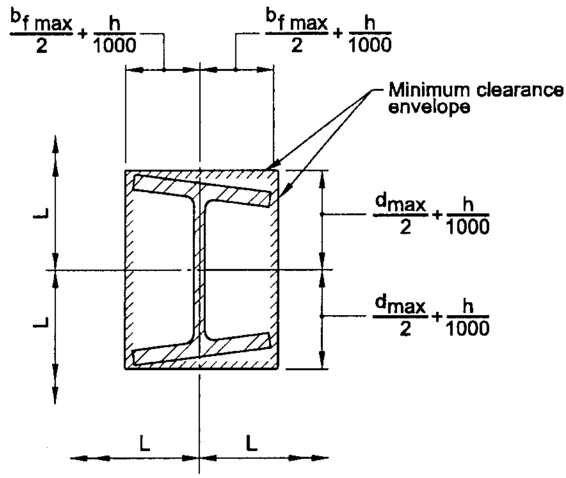
7.13.1.2. For members other than column shipping pieces, the following limitations shall apply:

(a) For a member that consists of an individual, straight shipping piece without field splices, other than a cantilevered member, the variation in alignment shall be acceptable if it is caused solely by variations in column alignment and/or primary supporting member alignment that are within the permissible variations for the fabrication and erection of such members.

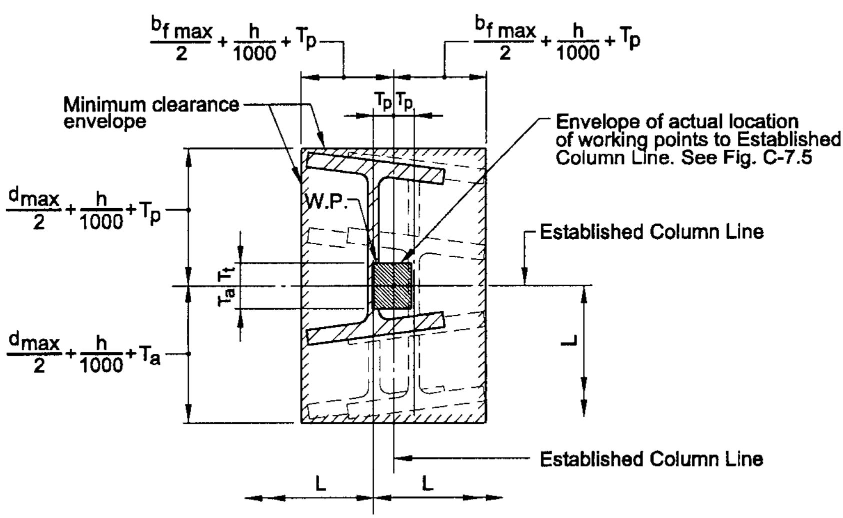
(b) For a member that consists of an individual, straight shipping piece that connects to a column, the variation in the distance from the member working point to the upper finished splice line of the column shall be equal to or less than plus 3/16 in. [5 mm] and minus 5/16 in. [8 mm].

(c) For a member that consists of an individual shipping piece that does not connect to a column, the variation in elevation shall be acceptable if it is caused solely by the variations in the elevations of the supporting members within the permissible variations for the fabrication and erection of those members.

(d) For a member that consists of an individual, straight shipping piece and that is a segment of a field assembled unit containing field splices between points of support, the plumbness, elevation and alignment shall be acceptable if the angular variation of the working line from the plan alignment is equal to or less than 1/500 of the distance between working points.



For enclosures or attachments that may follow column alignment.



For enclosures or attachments that must be held to precise plan location.

L = Actual center to center of columns = plan dimensions ± column cross section tolerance of columns ± beam length tolerance.

T_a = Plumbness tolerance away from building line (varies, see Fig. C-7.5)

T_t = Plumbness tolerance toward building line (varies, see Fig. C-7.5)

T_p = Plumbness tolerance parallel to building line (=T_a)

Figure C-7.4. Clearance required to accommodate accumulated column tolerance.

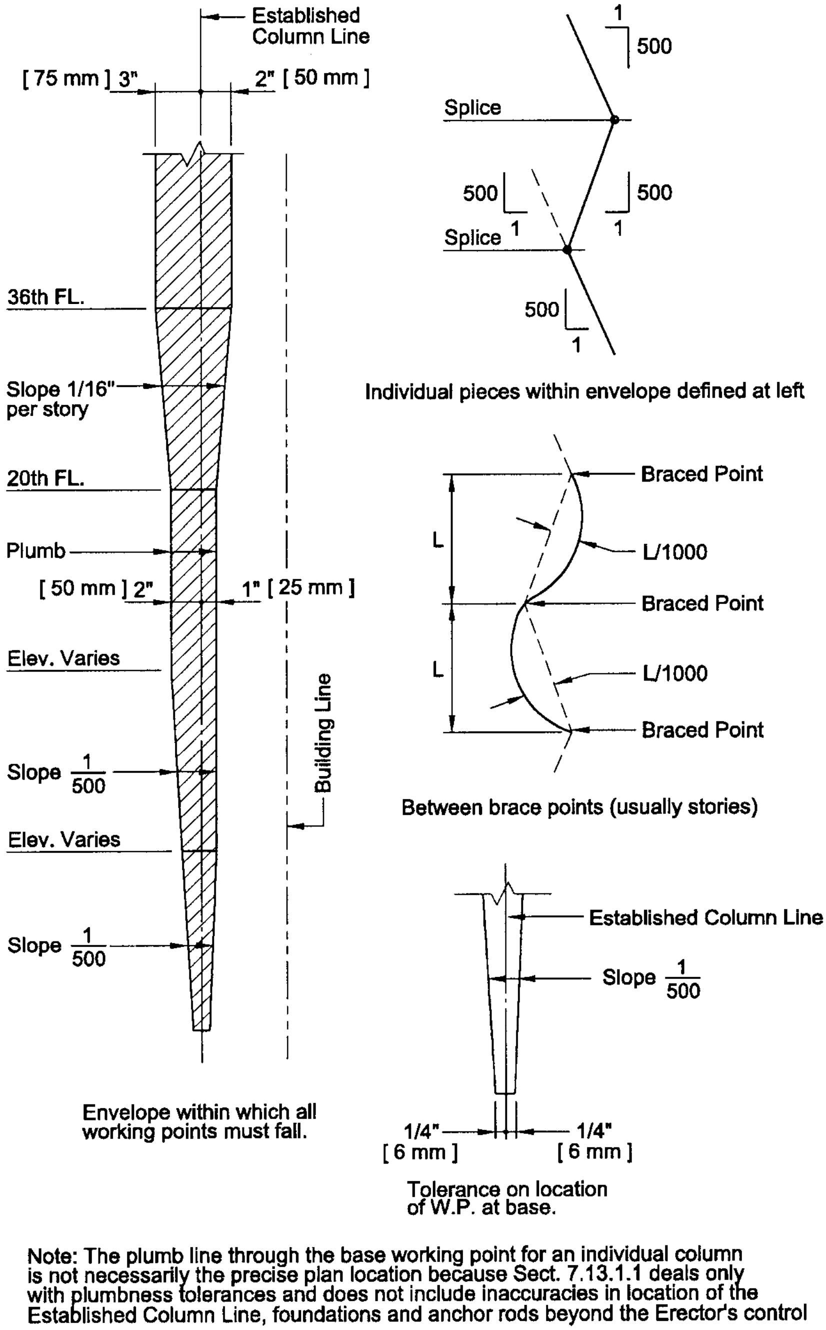


Figure C-7.5.Exterior column plumbness tolerances normal to building line.

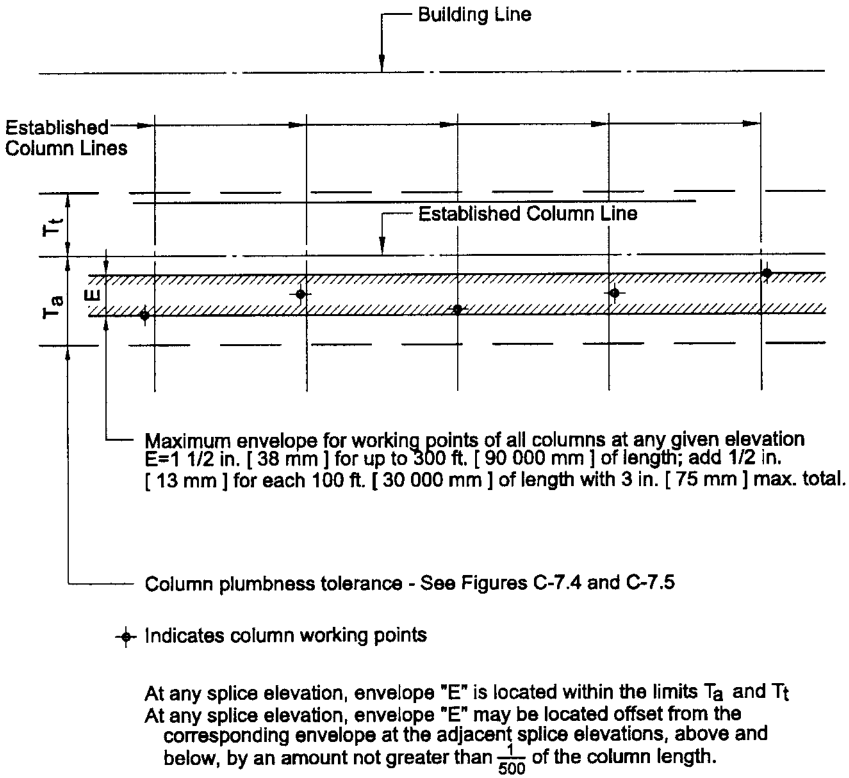


Figure C-7.6. Tolerances in plan at any splice elevation of exterior columns.

(e) For a cantilevered member that consists of an individual, straight shipping piece, the plumbness, elevation and alignment shall be acceptable if the angular variation of the working line from a straight line that is extended in the plan direction from the working point at its supported end is equal to or less than 1/500 of the distance from the working point at the free end.

(f) For a member of irregular shape, the plumbness, elevation and alignment shall be acceptable if the fabricated member is within its tolerances and the members that support it are within the tolerances specified in this Code.

Commentary:
The angular misalignment of the working line of all fabricated shipping pieces relative to the line between support points of the member as a whole in erected position must not exceed 1 in 500. Note that the tolerance is not stated in terms of a linear displacement at any point and is not to be taken as the overall length between supports divided by 500. Typical examples are

shown in Figure C–7.7. Numerous conditions within tolerance for these and other cases are possible. This condition applies to both plan and elevation tolerances.

(g) For a member that is fully assembled in the field in an unstressed condition, the same tolerances shall apply as if fully assembled in the shop.

(h) For a member that is field-assembled, element-by-element in place, temporary support shall be used or an alternative erection plan shall be submitted to the Owner's Designated Representatives for Design and Construction. The tolerance in Section 7.13.1.2(d) shall be met in the supported condition with working points taken at the point(s) of temporary support.

Commentary:
Trusses fabricated and erected as a unit or as an assembly of truss segments normally have excellent controls on vertical position regardless of fabrication and erection techniques. However, a truss fabricated and erected by assembling individual components in place in the field is potentially more sensitive to deflections of the individual truss components and the partially completed work during erection, particularly the chord members. In such a case, the erection process should follow an erection plan that addresses this issue.

7.13.1.3. For members that are identified as Adjustable Items by the Owner's Designated Representative for Design in the Contract Documents, the Fabricator shall provide adjustable Connections for these members to the supporting Structural Steel frame. Otherwise, the Fabricator is permitted to provide non-adjustable Connections. When Adjustable Items are specified, the Owner's Designated Representative for Design shall indicate the total adjustability that is required for the proper alignment of these supports for other trades. The variation in the position and alignment of Adjustable Items shall be as follows:

(a) The variation in the vertical distance from the upper finished splice line of the nearest column to the support location specified in the structural Design Drawings shall be equal to or less than plus or minus 3/8 in. [10 mm].
(b) The variation in the horizontal distance from the established finish line at the particular floor shall be equal to or less than plus or minus 3/8 in. [10 mm].
(c) The variation in vertical and horizontal alignment at the abutting ends of Adjustable Items shall be equal to or less than plus or minus 3/16 in. [5 mm].

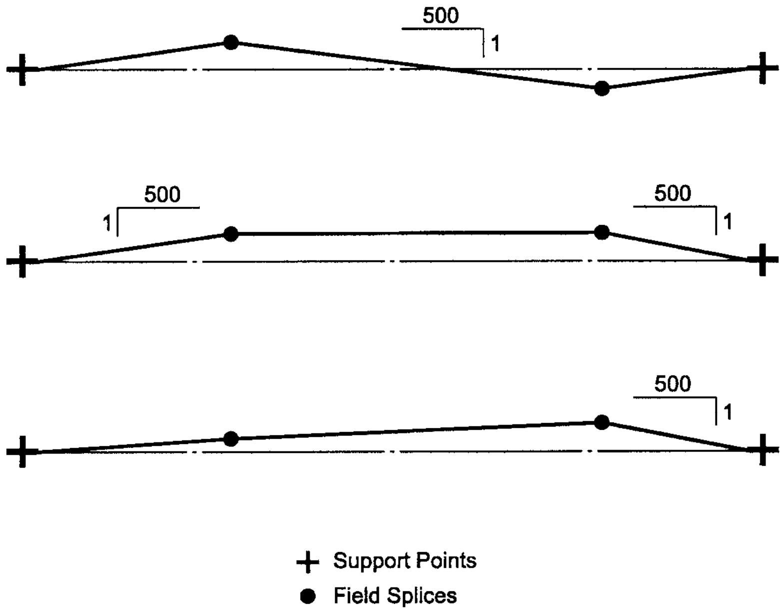


Figure C-7.7. Alignment tolerances for members with field splices.

Commentary:
When the alignment of lintels, wall supports, curb angles, mullions and similar supporting members for the use of other trades is required to be closer than that permitted by the foregoing tolerances for Structural Steel, the Owner's Designated Representative for Design must identify such items in the Contract Documents as Adjustable Items.

7.13.2. In the design of steel structures, the Owner's Designated Representative for Design shall provide for the necessary clearances and adjustments for material furnished by other trades to accommodate the mill tolerances, fabrication tolerances and erection tolerances in this Code for the Structural Steel frame.

Commentary:
In spite of all efforts to minimize inaccuracies, deviations will still exist; therefore, in addition, the designs of prefabricated wall panels, partition panels, fenestrations, floor-to-ceiling door frames and similar elements must provide for clearance and details for adjustment as described in Section 7.13.2. Designs must provide for adjustment in the vertical dimension of prefabricated facade panels that are supported by the Structural Steel frame because the accumulation of shortening of loaded steel columns will result in the unstressed facade supported at each floor level being higher than the Structural Steel framing to

which it must be attached. Observations in the field have shown that where a heavy facade is erected to a greater height on one side of a multistory building than on the other, the Structural Steel framing will be pulled out of alignment. Facades should be erected at a relatively uniform rate around the perimeter of the structure.

7.13.3. Prior to placing or applying any other materials, the Owner's Designated Representative for Construction shall determine that the location of the Structural Steel is acceptable for plumbness, elevation and alignment. The Erector shall be given either timely notice of acceptance by the Owner's Designated Representative for Construction, or a listing of specific items that are to be corrected in order to obtain acceptance. Such notice shall be rendered promptly upon completion of any part of the work and prior to the start of work by other trades that may be supported, attached or applied to the Structural Steel frame.

7.14. Correction of Errors

The correction of minor misfits by moderate amounts of reaming, grinding, welding or cutting, and the drawing of elements into line with drift pins, shall be considered to be normal erection operations. Errors that cannot be corrected using the foregoing means, or that require major changes in member or Connection configuration, shall be promptly reported to the Owner's Designated Representatives for Design and Construction and the Fabricator by the Erector, to enable the responsible entity to either correct the error or approve the most efficient and economical method of correction to be used by others.

Commentary:

As used in this Section, the term “moderate” refers to the amount of reaming, grinding, welding or cutting that must be done on the project as a whole, not the amount that is required at an individual location. It is not intended to address limitations on the amount of material that is removed by reaming at an individual bolt hole, for example, which is limited by the bolt-hole size and tolerance requirements in the AISC and RCSC Specifications.

7.15. Cuts, Alterations and Holes for Other Trades

Neither the Fabricator nor the Erector shall cut, drill or otherwise alter their work, nor the work of other trades, to accommodate other trades, unless such work is clearly specified in the Contract Documents. When such work is so specified, the Owner's Designated Representatives for Design and Construction shall furnish complete information as to materials, size, location and number of alterations in a timely manner so as not to delay the preparation of Shop and Erection Drawings.

7.16. Handling and Storage

The Erector shall take reasonable care in the proper handling and storage of the Structural Steel during erection operations to avoid the accumulation of excess dirt and foreign matter. The Erector shall not be responsible for the removal from the Structural Steel of dust, dirt or other foreign matter that may accumulate during erection as the result of job-site conditions or exposure to the elements. The Erector shall handle and store all bolts, nuts, washers and related fastening products in accordance with the requirements of the RCSC Specification.

Commentary:
During storage, loading, transport, unloading and erection, blemish marks caused by slings, chains, blocking, tie-downs, etc., occur in varying degrees. Abrasions caused by handling or cartage after painting are to be expected. It must be recognized that any shop-applied coating, no matter how carefully protected, will require touching-up in the field. Touching-up of these blemished areas is the responsibility of the contractor performing the field touch-up or field painting.

The Erector is responsible for the proper storage and handling of fabricated Structural Steel at the job site during erection. Shop-painted Structural Steel that is stored in the field pending erection should be kept free of the ground and positioned so as to minimize the potential for water retention. The Owner or Owner's Designated Representative for Construction is responsible for providing suitable job-site conditions and proper access so that the Fabricator/Erector may perform its work.

Job-site conditions are frequently muddy, sandy, dusty or a combination thereof during the erection period. Under such conditions it may be impossible to store and handle the Structural Steel in such a way as to completely avoid any accumulation of mud, dirt or sand on the surface of the Structural Steel, even though the Fabricator and the Erector manages to proceed with their work.

Repairs of damage to painted surfaces and/or removal of foreign materials due to adverse job-site conditions are outside the scope of responsibility of the Fabricator and the Erector when reasonable attempts at proper handling and storage have been made.

7.17. Field Painting

Neither the Fabricator nor the Erector is responsible to paint field bolt heads and nuts or field welds, nor to touch up abrasions of the shop coat, nor to perform any other field painting.

7.18. Final Cleaning Up

Upon the completion of erection and before final acceptance, the Erector shall remove all of the Erector's falsework, rubbish and temporary buildings.

SECTION 8. QUALITY ASSURANCE

8.1. General

8.1.1. The Fabricator shall maintain a quality assurance program to ensure that the work is performed in accordance with the requirements in this Code, the AISC Specification and the Contract Documents. The Fabricator shall have the option to use the AISC Quality Certification Program to establish and administer the quality assurance program.

Commentary:
The AISC Quality Certification Program confirms to the construction industry that a certified Structural Steel fabrication shop has the capability by reason of commitment, personnel, organization, experience, procedures, knowledge and equipment to produce fabricated Structural Steel of the required quality for a given category of work. The AISC Quality Certification Program is not intended to involve inspection and/or judgment of product quality on individual projects. Neither is it intended to guarantee the quality of specific fabricated Structural Steel products.

8.1.2. The Erector shall maintain a quality assurance program to ensure that the work is performed in accordance with the requirements in this Code, the AISC Specification and the Contract Documents. The Erector shall be capable of performing the erection of the Structural Steel, and shall provide the equipment, personnel and management for the scope, magnitude and required quality of each project. The Erector shall have the option to use the AISC Erector Certification Program to establish and administer the quality assurance program.

Commentary:
The AISC Erector Certification Program confirms to the construction industry that a certified Structural Steel Erector has the capability by reason of commitment, personnel, organization, experience, procedures, knowledge and equipment to erect fabricated Structural Steel to the required quality for a given category of work. The AISC Erector Certification Program is not intended to involve inspection and/or judgment of product quality on individual projects. Neither is it intended to guarantee the quality of specific erected Structural Steel products.

8.1.3. When the Owner requires more extensive quality assurance or independent inspection by qualified personnel, or requires that the Fabricator be certified under the AISC Quality Certification Program and/or requires that the Erector be certified under the AISC Erector Certification Program, this shall be clearly stated in the Contract Documents, including a definition of the scope of such inspection.

8.2. Inspection of Mill Material

Certified mill test reports shall constitute sufficient evidence that the mill product satisfies material order requirements. The Fabricator shall make a visual inspection of material that is received from the mill, but need not perform any material tests unless the Owner's Designated Representative for Design specifies in the Contract Documents that additional testing is to be performed at the Owner's expense.

8.3. Non-Destructive Testing

When non-destructive testing is required, the process, extent, technique and standards of acceptance shall be clearly specified in the Contract Documents.

8.4. Surface Preparation and Shop Painting Inspection

Inspection of surface preparation and shop painting shall be planned for the acceptance of each operation as the Fabricator completes it. Inspection of the paint system, including material and thickness, shall be made promptly upon completion of the paint application. When wet-film thickness is to be inspected, it shall be measured during the application.

8.5. Independent Inspection

When inspection by personnel other than those of the Fabricator and/or Erector is specified in the Contract Documents, the requirements in Sections 8.5.1 through 8.5.6 shall be met.

8.5.1. The Fabricator and the Erector shall provide the Inspector with access to all places where the work is being performed. A minimum of 24 hours notification shall be given prior to the commencement of work.

8.5.2. Inspection of shop work by the Inspector shall be performed in the Fabricator's shop to the fullest extent possible. Such inspections shall be timely, in-sequence and performed in such a manner as will not disrupt fabrication operations and will permit the repair of non-conforming work prior to any required painting while the material is still in-process in the fabrication shop.

8.5.3. Inspection of field work shall be promptly completed without delaying the progress or correction of the work.

8.5.4. Rejection of material or workmanship that is not in conformance with the Contract Documents shall be permitted at any time during the progress of the work. However, this provision shall not relieve the Owner or the Inspector of the obligation for timely, in-sequence inspections.

8.5.5. The Fabricator, Erector, and Owner's Designated Representatives for Design and Construction shall be informed of deficiencies that are noted by the Inspector promptly after the inspection. Copies of all reports prepared by the Inspector shall be promptly given to the Fabricator, Erector and Owner's Designated Representatives for Design and Construction. The necessary corrective work shall be performed in a timely manner.

8.5.6. The Inspector shall not suggest, direct, or approve the Fabricator or Erector to deviate from the Contract Documents or the approved Shop and Erection Drawings, or approve such deviation, without the written approval of the Owner's Designated Representatives for Design and Construction.

SECTION 9. CONTRACTS

9.1. Types of Contracts

9.1.1. For contracts that stipulate a lump sum price, the work that is required to be performed by the Fabricator and the Erector shall be completely defined in the Contract Documents.

9.1.2. For contracts that stipulate a price per pound, the scope of work that is required to be performed by the Fabricator and the Erector, the type of materials, the character of fabrication and the conditions of erection shall be based upon the Contract Documents, which shall be representative of the work to be performed.

9.1.3. For contracts that stipulate a price per item, the work that is required to be performed by the Fabricator and the Erector shall be based upon the quantity and the character of the items that are described in the Contract Documents.

9.1.4. For contracts that stipulate unit prices for various categories of Structural Steel, the scope of work that is required to be performed by the Fabricator and the Erector shall be based upon the quantity, character and complexity of the items in each category as described in the Contract Documents, and shall also be representative of the work to be performed in each category.

9.2. Calculation of Weights

Unless otherwise specified in the contract, for contracts stipulating a price per pound for fabricated Structural Steel that is delivered and/or erected, the quantities of materials for payment shall be determined by the calculation of the gross weight of materials as shown in the Shop Drawings.

Commentary:
The standard procedure for calculation of weights that is described in this Code meets the need for a universally acceptable system for defining "pay weights" in contracts based upon the weight of delivered and/or erected materials. These procedures permits the Owner to easily and accurately evaluate price-per-pound proposals from potential suppliers and enables all parties to a contract to have a clear and common understanding of the basis for payment.

The procedure in this Code affords a simple, readily understood method of calculation that will produce pay weights that are consistent throughout the industry and that may be easily verified by the Owner. While this procedure does not produce actual weights, it can be used by purchasers and suppliers to define a widely accepted basis for bidding and contracting for Structural Steel. However, any other system can be used as the basis for a contractual agreement. When other systems are used, both the supplier and the purchaser should clearly understand how the alternative procedure is handled.

9.2.1. The unit weight of steel shall be taken as 490 lb/ft^3 [7 850 kg/m^3]. The unit weight of other materials shall be in accordance with the manufacturer's published data for the specific product.

9.2.2. The weights of Standard Structural Shapes, plates and bars shall be calculated on the basis of Shop Drawings that show the actual quantities and dimensions of material to be fabricated, as follows:

(a) The weights of all Standard Structural Shapes shall be calculated using the nominal weight per ft [mass per m] and the detailed overall length.
(b) The weights of plates and bars shall be calculated using the detailed overall rectangular dimensions.
(c) When parts can be economically cut in multiples from material of larger dimensions, the weight shall be calculated on the basis of the theoretical rectangular dimensions of the material from which the parts are cut.
(d) When parts are cut from Standard Structural Shapes, leaving a non-standard section that is not useable on the same contract, the weight shall be calculated using the nominal weight per ft [mass per m] and the overall length of the Standard Structural Shapes from which the parts are cut.
(e) Deductions shall not be made for material that is removed for cuts, copes, clips, blocks, drilling, punching, boring, slot milling, planing or weld joint preparation.

9.2.3. The items for which weights are shown in tables in the AISC Manual of Steel Construction shall be calculated on the basis of the tabulated weights shown therein.

9.2.4. The weights of items that are not shown in tables in the AISC Manual of Steel Construction shall be taken from the manufacturer's catalog and the manufacturer's shipping weight shall be used.

Commentary:
Many items that are weighed for payment purposes are not tabulated with weights in the AISC Manual of Steel Construction. These include, but are not limited to, Anchor Rods, clevises, turnbuckles, sleeve nuts, recessed-pin nuts, cotter pins and similar devices.

9.2.5. The weights of shop or field weld metal and protective coatings shall not be included in the calculated weight for the purposes of payment.

9.3. Revisions to the Contract Documents

Revisions to the Contract Documents shall be confirmed by change order or extra work order. Unless otherwise noted, the issuance of a revision to the

Contract Documents shall constitute authorization by the Owner that the revision is Released for Construction. The contract price and schedule shall be adjusted in accordance with Sections 9.4 and 9.5.

9.4. Contract Price Adjustment

9.4.1. When the scope of work and responsibilities of the Fabricator and the Erector are changed from those previously established in the Contract Documents, an appropriate modification of the contract price shall be made. In computing the contract price adjustment, the Fabricator and the Erector shall consider the quantity of work that is added or deleted, the modifications in the character of the work and the timeliness of the change with respect to the status of material ordering, detailing, fabrication and erection operations.

Commentary:
The fabrication and erection of Structural Steel is a dynamic process. Typically, material is being acquired at the same time that the Shop and Erection Drawings are being prepared. Additionally, the fabrication shop will normally fabricate pieces in the order that the Structural Steel is being shipped and erected.

Items that are revised or placed on hold generally upset these relationships and can be very disruptive to the detailing, fabricating and erecting processes. The provisions in Sections 3.5, 4.4.2 and 9.3 are intended to minimize these disruptions so as to allow work to continue. Accordingly, it is required in this Code that the reviewer of requests for contract price adjustments recognize this and allow compensation to the Fabricator and the Erector for these inefficiencies and for the materials that are purchased and the detailing, fabrication and erection that has been performed, when affected by the change.

9.4.2. Requests for contract price adjustments shall be presented by the Fabricator and/or the Erector in a timely manner and shall be accompanied by a description of the change that is sufficient to permit evaluation and timely approval by the Owner.

9.4.3. Price-per-pound and price-per-item contracts shall provide for additions or deletions to the quantity of work that are made prior to the time the work is Released for Construction. When changes are made to the character of the work at any time, or when additions and/or deletions are made to the quantity of the work after it is released for detailing, fabrication or erection, the contract price shall be equitably adjusted.

9.5. Scheduling

9.5.1. The contract schedule shall state when the Design Drawings will be Released for Construction, if the Design Drawings are not available at the time of bidding, and when the job site, foundations, piers and abutments will be ready, free from obstructions and accessible to the Erector, so that erection can start at the designated time and continue without interference or delay caused by the Owner's Designated Representative for Construction or other trades.

9.5.2. The Fabricator and the Erector shall advise the Owner's Designated Representatives for Design and Construction, in a timely manner, of the effect any revision has on the contract schedule.

9.5.3. If the fabrication or erection is significantly delayed due to revisions to the requirements of the contract, or for other reasons that are the responsibility of others, the Fabricator and/or Erector shall be compensated for the additional costs incurred.

9.6. Terms of Payment

The Fabricator shall be paid for Mill Materials and fabricated product that is stored off the job site. Other terms of payment for the contract shall be outlined in the Contract Documents.

Commentary:
These terms include such items as progress payments for material, fabrication, erection, retainage, performance and payment bonds and final payment. If a performance or payment bond, paid for by the Owner, is required by contract, no retainage shall be required.

SECTION 10. ARCHITECTURALLY EXPOSED STRUCTURAL STEEL

10.1. General Requirements

When members are specifically designated as "Architecturally Exposed Structural Steel" or "AESS" in the Contract Documents, the requirements in Sections 1 through 9 shall apply as modified in Section 10. AESS members or components shall be fabricated and erected with the care and dimensional tolerances that are stipulated in Sections 10.2 through 10.4. The following additional information shall be provided in the Contract Documents when AESS is specified:

(a) Specific identification of members or components that are AESS;
(b) Fabrication and/or erection tolerances that are to be more restrictive than provided for in this Section, if any; and,
(c) Requirements, if any, of a mock-up panel or components for inspection and acceptance standards prior to the start of fabrication.

Commentary:
This Section of this Code defines additional requirements that apply only to members that are specifically designated by the Contract Documents as "Architecturally Exposed Structural Steel" (AESS). The rapidly increasing use of exposed Structural Steel as a medium of architectural expression has given rise to a demand for closer dimensional tolerances and smoother finished surfaces than required for ordinary Structural Steel framing.

This Section of this Code establishes standards for these requirements that take into account both the desired finished appearance and the abilities of the fabrication shop to produce the desired product. It should be pointed out that the term "Architecturally Exposed Structural Steel" (AESS), as covered in this Section, must be specified in the Contract Documents if the Fabricator is required to meet the fabricating standards in this Section, and applies only to that portion of the Structural Steel so identified.

AESS requirements usually involve significant cost in excess of that for Structural Steel that is fabricated in the absence of an AESS requirement. Therefore, the designation AESS should be applied rationally, with visual acceptance criteria that are appropriate for the distance at which the exposed element will be viewed in the completed structure. In order to avoid misunderstandings and to hold costs to a minimum, only those Structural Steel surfaces and Connections that will remain exposed and subject to normal view by pedestrians or occupants of the completed structure should be designated as AESS.

10.2. Fabrication

10.2.1. The permissible tolerances for out-of-square or out-of-parallel, depth, width and symmetry of rolled shapes shall be as specified in ASTM A6/A6M. Unless otherwise specified in the Contract Documents, the exact matching of abutting cross-sectional configurations shall not be necessary. The as-fabricated straightness tolerances of members shall be one-half of the standard camber and sweep tolerances in ASTM A6/A6M.

10.2.2. The tolerances on overall profile dimensions of members that are built-up from a series of Standard Structural Shapes, plates and/or bars by welding shall be taken as the accumulation of the variations that are permitted for the component parts in ASTM A6/A6M. The as-fabricated straightness tolerances for the member as a whole shall be one-half the standard camber and sweep tolerances for rolled shapes in ASTM A6/A6M.

10.2.3. Unless specific visual acceptance criteria for Weld Show-Through are specified in the Contract Documents, the members or components shall be acceptable as produced.

Commentary:
Weld Show-Through is generally a function of weld size and material thickness.

10.2.4. All copes, miters and cuts in surfaces that are exposed to view shall be made with uniform gaps of 1/8 in. [3 mm] if shown as open joints, or in reasonable contact if shown without gap.

10.2.5. All welds that are exposed to view shall be visually acceptable if they meet the requirements in AWS D1.1, except all groove and plug welds that are exposed to view shall not project more than 1/16 in. [2 mm] above the exposed surface. Finishing or grinding of welds shall not be necessary, unless such treatment is required to provide for clearances or fit of other components.

10.2.6. Erection marks or other painted marks shall not be made on those surfaces of weathering steel AESS members that are to be exposed in the completed structure. Unless otherwise specified in the Contract Documents, the Fabricator shall clean weathering steel AESS members to meet the requirements of SSPC-SP6.

10.2.7. Stamped or raised manufacturer's identification marks shall not be filled, ground or otherwise removed.

10.2.8. Seams of hollow structural sections shall be acceptable as produced. Seams shall be oriented away from view or as directed in the Contract Documents.

10.3. Delivery of Materials

The Fabricator shall use special care to avoid bending, twisting or otherwise distorting the Structural Steel.

10.4. Erection

10.4.1. The Erector shall use special care in unloading, handling and erecting the Structural Steel to avoid marking or distorting the Structural Steel. Care shall also be taken to minimize damage to any shop paint. If temporary braces or erection clips are used, care shall be taken to avoid the creation of unsightly surfaces upon removal. Tack welds shall be ground smooth and holes shall be filled with weld metal or body solder and smoothed by grinding or filing. The Erector shall plan and execute all operations in such a manner that the close fit and neat appearance of the structure will not be impaired.

10.4.2. Unless otherwise specified in the Contract Documents, AESS members and components shall be plumbed, leveled and aligned to a tolerance that is one-half that permitted for non-AESS members. To accommodate these erection tolerances for AESS, the Owner's Designated Representative for Design shall specify Connections between AESS members and non-AESS members, masonry, concrete and other supports as Adjustable Items, in order to provide the Erector with means for adjustment.

10.4.3. When AESS is backed with concrete, the Owner's Designated Representative for Construction shall provide sufficient shores, ties and strongbacks to prevent sagging, bulging or similar deformation of the AESS members due to the weight and pressure of the wet concrete.

APPENDIX A. DIGITAL BUILDING PRODUCT MODELS

The provisions in this Appendix shall apply when the contract documents indicate that a three-dimensional digital building product model replaces contract drawings and is to be used as the primary means of designing, representing, and exchanging structural steel data for the project. When this is the case, all references to the Design Drawings in this Code shall instead apply to the Design Model, and all references to the Shop and Erection Drawings in the Code shall instead apply to the Manufacturing Model. The CIS/2 Logical Product Model shall be used as the building product model for structural steel.

If the primary means of project communication reverts from a model-based system to a paper-based system, the requirements in this Code other than in this Appendix shall apply.

Commentary:
Current technology permits the transfer of three-dimensional digital building product model data among the design and construction teams for a project. Over the last several years, designers and fabricators have used CIS/2 as a standard format in the exchange of building product models representing the steel structure. This Appendix facilitates the use of this technology in the design and construction of steel structures, and eliminates any interpretation of this Code that might be construed to prohibit or inhibit the use of this technology. While the technology is new and there is no long-established standard of practice, it is the intent in this Appendix to provide guidance for its use.

APPENDIX A. GLOSSARY

Add the following definitions to the Glossary:

Building Product Model. A digital information structure of the objects making up a building, capturing the form, function, behavior and relations of the parts and assemblies within one or more building systems. A building product model can be implemented in multiple ways, including as an ASCII file or as a database. The data in the model is created, manipulated, evaluated, reviewed and presented using computer-based design, engineering, and manufacturing applications. Traditional two-dimensional drawings may be one of many reports generated by the building product model (see Eastman, Charles M.: Building Product Models: Computer Environments Supporting Design and Construction; 1999 by CRC Press).

CIS/2 (CIMSteel Integration Standards/Version 2). The specification providing the building product model for structural steel and format for electronic data interchange (EDI) among software applications dealing with steel design, analysis, and manufacturing.

Logical Product Model (LPM). The CIS/2 building product model, which supports the engineering of low-, medium- and high-rise construction, in domestic, commercial and industrial contexts. All elements of the structure are covered, including main and secondary framing and connections. The components used can be of any variety of structural shape or element.

The LPM addresses the exchange of data between structural steel applications. It is meant to support a heterogeneous set of applications over a fairly broad portion of the steel lifecycle. It is organized around three different sub-models: the Analysis Model (data represented in structural analysis), the Design Model (data represented in frame design layout) and the Manufacturing Model (data represented in detailing for fabrication).

Data Management Conformance (DMC). The capability of the CIMSteel model to include optional data entities for managing and tracking additions, deletions and modifications to a model, including who made the change and when the change was made for all data changes.

A1.2. Referenced Specifications, Codes and Standards

Add the following reference to Section 1.2:

CIMSteel Integration Standards Release 2: Second Edition P265: CIS/2.1: Volumes 1 through 4.

A3. DESIGN DRAWINGS AND SPECIFICATIONS

In addition to the requirements in Section 3, the following requirements shall apply to the Design Model:

A3.1. Design Model

The Design Model shall:

(a) Consist of Data Management Conformance Classes.
(b) Contain Analysis Model data so as to include load calculations as specified in the Contract Documents.
(c) Include entities that fully define each steel element and the extent of detailing of each element, as would be recorded on equivalent set of structural steel design drawings.
(d) Include all steel elements identified in the Contract Documents as well as any other entities required for strength and stability of the completely erected structure.
(e) Govern over all other forms of information, including drawings, sketches, etc.

A3.2. LPM Administration

The Owner shall designate an Administrator for the LPM, who shall:

(a) Control the LPM by providing appropriate access privileges (read, write, etc) to all relevant parties.
(b) Maintain the security of the LPM.
(c) Guard against data loss of the LPM.
(d) Be responsible for updates and revisions to the LPM as they occur.
(e) Inform all appropriate parties as to changes to the LPM.

Commentary:
When a project is designed and constructed using EDI, it is imperative that an individual entity on the team be responsible for maintaining the LPM. This is to assure protection of data through proper backup, storage and security and to provide coordination of the flow of information to all team members when information is added to the model. Team members exchange information to revise the model with this Administrator. The Administrator will validate all changes to the LPM. This is to assure proper tracking and control of revisions.

This Administrator can be one of the design team members such as an Architect, Structural Engineer or a separate entity on the design team serving this purpose. The Administrator can also be the Fabricator's detailer or a separate entity on the construction team serving this purpose.

A4.3. Fabricator Responsibility

In addition to the requirements in Section 4.3, the following requirements shall apply:

When the Design Model is used to develop the Manufacturing Model the fabricator shall accept the information under the following conditions:

(a) When the design information is to be conveyed to the Fabricator by way of the Design Model, in the event of a conflict between the model and the Design Drawings, the Design Model will control.
(b) The ownership of the information added to the LPM in the Manufacturing Model should be defined in the Contract Documents. In the absence of terms for ownership regarding the information added by the Fabricator to the LPM in the Contract Documents, the ownership will belong to the Fabricator.
(c) During the development of the Manufacturing Model, as member locations are adjusted to convert the modeled parts from a Design Model, these relocations will only be done with the approval of the Owner's Designated Representative for Design.
(d) The Fabricator and Erector shall accept the use of the LPM and Design Model under the same conditions as set forth in Paragraph 4.3 with regard to CAD files, except as modified in A4.3 above.

A4.4. Approval

In addition to the requirements in Section 4.4, the following requirements shall apply:

When the approval of the detailed material is to be done by the use of the Manufacturing Model the version of the submitted model shall be identified. The approver shall annotate the Manufacturing Model with approval comments attached to the individual elements as specified in the CIS/2 standard. As directed by the approval comment the Fabricator will reissue the Manufacturing Model for re-review and the version of the model submitted will be tracked as previously defined.

Commentary:
Approval of the Manufacturing Model by the Owner's Designated Representative for Design can replace the approval of actual shop and erection drawings. For this method to be effective, a system must be in place to record review, approval, correction and final release of the Manufacturing Model for fabrication of structural steel. The versions of the model must be tracked, and review comments and approvals permanently attached to the versions of the model to the same extent as such data is maintained with conventional hard copy approvals. The CIS/2 standard provides this level of tracking.

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AISC 303-05 (2M505)

PART 17

MISCELLANEOUS DATA AND MATHEMATICAL INFORMATION

Table 17–1
SI Equivalents of Standard U.S. Shape Profiles
W Shapes

Shape	SI Equivalent
in. × lb/ft	mm × kg/m
W44×335	W1100×499
×290	×433
×262	×390
×230	×343
W40×593	W1000×883
×503	×748
×431	×642
×397	×591
×372	×554
×362	×539
×324	×483
×297	×443
×277	×412
×249	×371
×215	×321
×199	×296
W40×392	W1000×584
×331	×494
×327	×486
×294	×438
×278	×415
×264	×393
×235	×350
×211	×314
×183	×272
×167	×249
×149	×222
W36×800	W920×1191
×652	×970
×529	×787
×487	×725
×441	×656
×395	×588
×361	×537
×330	×491
×302	×449
×282	×420
×262	×390
×247	×368
×231	×345

Shape	SI Equivalent
in. × lb/ft	mm × kg/m
W36×256	W920×381
×232	×345
×210	×313
×194	×289
×182	×271
×170	×253
×160	×238
×150	×223
×135	×201
W33×387	W840×576
×354	×527
×318	×473
×291	×433
×263	×392
×241	×359
×221	×329
×201	×299
W33×169	W840×251
×152	×226
×141	×210
×130	×193
×118	×176
W30×391	W760×582
×357	×531
×326	×484
×292	×434
×261	×389
×235	×350
×211	×314
×191	×284
×173	×257
W30×148	W760×220
×132	×196
×124	×185
×116	×173
×108	×161
×99	×147
×90	×134

Shape	SI Equivalent
in. × lb/ft	mm × kg/m
W27×539	W690×802
×368	×548
×336	×500
×307	×457
×281	×419
×258	×384
×235	×350
×217	×323
×194	×289
×178	×265
×161	×240
×146	×217
W27×129	W690×192
×114	×170
×102	×152
×94	×140
×84	×125
W24×370	W610×551
×335	×498
×306	×455
×279	×415
×250	×372
×229	×341
×207	×307
×192	×285
×176	×262
×162	×241
×146	×217
×131	×195
×117	×174
×104	×155
W24×103	W610×153
×94	×140
×84	×125
×76	×113
×68	×101
W24×62	W610×92
×55	×82

Table 17–1 (continued)
SI Equivalents of Standard U.S. Shape Profiles
W Shapes

Shape	SI Equivalent
in. × lb/ft	mm × kg/m
W21×201	W530×300
×182	×272
×166	×248
×147	×219
×132	×196
×122	×182
×111	×165
×101	×150
W21×93	W530×138
×83	×123
×73	×109
×68	×101
×62	×92
×55	×82
×48	×72
W21×57	W530×85
×50	×74
×44	×66
W18×311	W460×464
×283	×421
×258	×384
×234	×349
×211	×315
×192	×286
×175	×260
×158	×235
×143	×213
×130	×193
×119	×177
×106	×158
×97	×144
×86	×128
×76	×113
W18×71	W460×106
×65	×97
×60	×89
×55	×82
×50	×74
W18×46	W460×68
×40	×60
×35	×52

Shape	SI Equivalent
in. × lb/ft	mm × kg/m
W16×100	W410×149
×89	×132
×77	×114
×67	×100
W16×57	W410×85
×50	×75
×45	×67
×40	×60
×36	×53
W16×31	W410×46.1
×26	×38.8
W14×730	W360×1086
×665	×990
×605	×900
×550	×818
×500	×744
×455	×677
×426	×634
×398	×592
×370	×551
×342	×509
×311	×463
×283	×421
×257	×382
×233	×347
×211	×314
×193	×287
×176	×262
×159	×237
×145	×216
W14×132	W360×196
×120	×179
×109	×162
×99	×147
×90	×134
W14×82	W360×122
×74	×110
×68	×101
×61	×91

Shape	SI Equivalent
in. × lb/ft	mm × kg/m
W14×53	W360×79
×48	×72
×43	×64
W14×38	W360×58
×34	×51
×30	×44.6
W14×26	W360×39
×22	×32.9
W12×336	W310×500
×305	×454
×279	×415
×252	×375
×230	×342
×210	×313
×190	×283
×170	×253
×152	×226
×136	×202
×120	×179
×106	×158
×96	×143
×87	×129
×79	×117
×72	×107
×65	×97
W12×58	W310×86
×53	×79
W12×50	W310×74
×45	×67
×40	×60
W12×35	W310×52
×30	×44.5
×26	×38.7
W12×22	W310×32.7
×19	×28.3
×16	×23.8
×14	×21.0

Table 17–1 (continued)
SI Equivalents of Standard U.S. Shape Profiles
W Shapes

Shape	SI Equivalent	Shape	SI Equivalent	Shape	SI Equivalent
in. × lb/ft	mm × kg/m	in. × lb/ft	mm × kg/m	in. × lb/ft	mm × kg/m
W10×112	W250×167	W10×19	W250×28.4	W8×15	W200×22.5
×100	×149	×17	×25.3	×13	×19.3
×88	×131	×15	×22.3	×10	×15.0
×77	×115	×12	×17.9	W6×25	W150×37.1
×68	×101	W8×67	W200×100	×20	×29.8
×60	×89	×58	×86	×15	×22.5
×54	×80	×48	×71	W6×16	W150×24.0
×49	×73	×40	×59	×12	×18.0
W10×45	W250×67	×35	×52	×9	×13.5
×39	×58	×31	×46.1	×8.5	×13.0
×33	×49.1	W8×28	W200×41.7	W5×19	W130×28.1
W10×30	W250×44.8	×24	×35.9	×16	×23.8
×26	×38.5	W8×21	W200×31.3	W4×13	W100×19.3
×22	×32.7	×18	×26.6		

Table 17–2
SI Equivalents of Standard U.S. Shape Profiles
M, S and HP Shapes

Shape	SI Equivalent	Shape	SI Equivalent	Shape	SI Equivalent
in. × lb/ft	mm × kg/m	in. × lb/ft	mm × kg/m	in. × lb/ft	mm × kg/m
M12.5×12.4	M318×18.5	S24×121	S610×180	HP14×117	HP360×174
×11.6	×17.3	×106	×158	×102	×152
				×89	×132
M12×11.8	M310×17.6	S24×100	S610×149	×73	×108
×10.8	×16.1	×90	×134		
		×80	×119	HP12×84	HP310×125
M12×10	M310×14.9			×74	×110
		S20×96	S510×143	×63	×93
M10×9	M250×13.4	×86	×128	×53	×79
×8	×11.9				
		S20×75	S510×112	HP10×57	HP250×85
M10×7.5	M250×11.2	×66	×98	×42	×62
M8×6.5	M200×9.7				
×6.2	×9.2	S18×70	S460×104	HP8×36	HP200×53
		×54.7	×81.4		
M6×4.4	M150×6.6				
×3.7	×5.5	S15×50	S380×74		
		×42.9	×64		
M5×18.9	M130×28.1				
		S12×50	S310×74		
M4×6	M100×8.9	×40.8	×60.7		
×4.08	×6.1				
×3.45	×5.1	S12×35	S310×52		
×3.2	×4.8	×31.8	×47.3		
		S10×35	S250×52		
M3×2.9	M75×4.3	×25.4	×37.8		
		S8×23	S200×34		
		×18.4	×27.4		
		S6×17.2	S150×25.7		
		×12.5	×18.6		
		S5×10	S130×15		
		S4×9.5	S100×14.1		
		×7.7	×11.5		
		S3×7.5	S75×11.2		
		×5.7	×8.5		

Table 17–3
SI Equivalents of Standard U.S. Shape Profiles
Channels

Shape	SI Equivalent	Shape	SI Equivalent
in. × lb/ft	mm × kg/m	in. × lb/ft	mm × kg/m
C15×50	C380×74	MC18×58	MC460×86
×40	×60	×51.9	×77.2
×33.9	×50.4	×45.8	×68.2
C12×30	C310×45	×42.7	×63.5
×25	×37	MC13×50	MC330×74
×20.7	×30.8	×40	×60
C10×30	C250×45	×35	×52
×25	×37	×31.8	×47.3
×20	×30	MC12×50	MC310×74
×15.3	×22.8	×45	×67
C9×20	C230×30	×40	×60
×15	×22	×35	×52
×13.4	×19.9	×31	×46
C8×18.5	C200×27.9	MC12×10.6	MC310×15.8
×13.7	×20.5	MC10×41.1	MC250×61.2
×11.5	×17.1	×33.6	×50
C7×14.7	C180×22	×28.5	×42.4
×12.2	×18.2	MC10×25	MC250×37
×9.8	×14.6	×22	×33
C6×13	C150×19.3	MC10×8.4	MC250×12.5
×10.5	×15.6	×6.5	×9.7
×8.2	×12.2	MC9×25.4	MC230×37.8
C5×9	C130×13	×23.9	×35.6
×6.7	×10.4	MC8×22.8	MC200×33.9
C4×7.2	C100×10.8	×21.4	×31.8
×5.4	×8	MC8×20	MC200×29.8
×4.5	×6.7	×18.7	×27.8
C3×6	C75×8.9	MC8×8.5	MC200×12.6
×5	×7.4	MC7×22.7	MC180×33.8
×4.1	×6.1	×19.1	×28.4
×3.5	×5.2	MC6×18	MC150×26.8
		×15.3	×22.8
		MC6×16.3	MC150×24.3
		×15.1	×22.5
		MC6×12	MC150×17.9
		MC6×7	MC150×10.4
		×6.5	×9.7
		MC4×13.8	MC100×20.5
		MC3×7.1	MC75×10.6

Table 17–4
SI Equivalents of Standard U.S. Shape Profiles
Angles

Shape	SI Equivalent
in. × in. × in.	mm × mm × mm
L8×8×1 1/8	L203×203×28.6
×1	×25.4
×7/8	×22.2
×3/4	×19.0
×5/8	×15.9
×9/16	×14.3
×1/2	×12.7
L8×6×1	L203×152×25.4
×7/8	×22.2
×3/4	×19.0
×5/8	×15.9
×9/16	×14.3
×1/2	×12.7
×7/16	×11.1
L8×4×1	L203×102×25.4
×7/8	×22.2
×3/4	×19.0
×5/8	×15.9
×9/16	×14.3
×1/2	×12.7
×7/16	×11.1
L7×4×3/4	L178×102×19.0
×5/8	×15.9
×1/2	×12.7
×7/16	×11.1
×3/8	×9.5
L6×6×1	L152×152×25.4
×7/8	×22.2
×3/4	×19.0
×5/8	×15.9
×9/16	×14.3
×1/2	×12.7
×7/16	×11.1
×3/8	×9.5
×5/16	×7.9
L6×4×7/8	L152×102×22.2
×3/4	×19.0
×5/8	×15.9
×9/16	×14.3
×1/2	×12.7
×7/16	×11.1
×3/8	×9.5
×5/16	×7.9
L6×3 1/2×1/2	L152×89×12.7
×3/8	×9.5
×5/16	×7.9
L5×5×7/8	L127×127×22.2
×3/4	×19.0
×5/8	×15.9
×1/2	×12.7
×7/16	×11.1
×3/8	×9.5
×5/16	×7.9
L5×3 1/2×3/4	L127×89×19.0
×5/8	×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
L5×3×1/2	L127×76×12.7
×7/16	×11.1
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
L4×4×3/4	L102×102×19
×5/8	×15.9
×1/2	×12.7
×7/16	×11.1
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
L4×3 1/2×1/2	L102×89×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
L4×3×5/8	L102×76×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
L3 1/2×3 1/2×1/2	L89×89×12.7
×7/16	×11.1
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
L3 1/2×3×1/2	L89×76×12.7
×7/16	×11.1
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
L3 1/2×2 1/2×1/2	L89×64×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
L3×3×1/2	L76×76×12.7
×7/16	×11.1
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
L3×2 1/2×1/2	L76×64×12.7
×7/16	×11.1
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8

Table 17–4 (continued)
SI Equivalents of Standard U.S. Shape Profiles
Angles

Shape in. × in. × in.	SI Equivalent mm × mm × mm	Shape in. × in. × in.	SI Equivalent mm × mm × mm	Shape in. × in. × in.	SI Equivalent mm × mm × mm
L3×2×1/2	L76×51×12.7	L2 1/2×2×3/8	L64×51×9.5	L2×2×3/8	L51×51×9.5
×3/8	×9.5	×5/16	×7.9	×5/16	×7.9
×5/16	×7.9	×1/4	×6.4	×1/4	×6.4
×1/4	×6.4	×3/16	×4.8	×3/16	×4.8
×3/16	×4.8	L2 1/2×1 1/2×1/4	L64×38×6.4	×1/8	×3.2
L2 1/2×2 1/2×1/2	L64×64×12.7	×3/16	×4.8		
×3/8	×9.5				
×5/16	×7.9				
×1/4	×6.4				
×3/16	×4.8				

Table 17–5
SI Equivalents of Standard U.S. Shape Profiles
WT Shapes

Shape	SI Equivalent
in. × lb/ft	mm × kg/m
WT22×167.5	WT550×249.5
×145	×216.5
×131	×195
×115	×171.5
WT20×296.5	WT500×441.5
×251.5	×374
×215.5	×321
×198.5	×295.5
×186	×277
×181	×269.5
×162	×241.5
×148.5	×221.5
×138.5	×206
×124.5	×185.5
×107.5	×160.5
×99.5	×148
WT20×196	WT500×292
×165.5	×247
×163.5	×243
×147	×219
×139	×207.5
×132	×196.5
×117.5	×175
×105.5	×157
×91.5	×136
×83.5	×124.5
×74.5	×111
WT18×400	WT460×595.5
×326	×485
×264.5	×393.5
×243.5	×362.5
×220.5	×328
×197.5	×294
×180.5	×268.5
×165	×245.5
×151	×224.5
×141	×210
×131	×195
×123.5	×184
×115.5	×172.5

Shape	SI Equivalent
in. × lb/ft	mm × kg/m
WT18×128	WT460×190.5
×116	×172.5
×105	×156.5
×97	×144.5
×91	×135.5
×85	×126.5
×80	×119
×75	×111.5
×67.5	×100.5
WT16.5×193.5	WT420×288
×177	×263.5
×159	×236.5
×145.5	×216.5
×131.5	×196
×120.5	×179.5
×110.5	×164.5
×100.5	×149.5
WT16.5×84.5	WT460×125.5
×76	×113
×70.5	×105
×65	×96.5
×59	×88
WT15×195.5	WT380×291
×178.5	×265.5
×163	×242
×146	×217
×130.5	×194.5
×117.5	×175
×105.5	×157
×95.5	×142
WT15×86.5	WT380×128.5
×74	×110
×66	×98
×62	×92.5
×58	×86.5
×54	×80.5
×49.5	×73.5
×45	×67

Shape	SI Equivalent
in. × lb/ft	mm × kg/m
WT13.5×269.5	WT345×401
×184	×274
×168	×250
×153.5	×228.5
×140.5	×209.5
×129	×192
×117.5	×175
×108.5	×161.5
×97	×144.5
×89	×132.5
×80.5	×120
×73	×108.5
WT13.5×64.5	WT345×96
×57	×85
×51	×76
×47	×70
×42	×62.5
WT12×185	WT305×275.5
×167.5	×249
×153	×227.5
×139.5	×207.5
×125	×186
×114.5	×170.5
×103.5	×153.5
×96	×142.5
×88	×131
×81	×120.5
×73	×108.5
×65.5	×97.5
×58.5	×87
×52	×77.5
WT12×51.5	WT305×76.5
×47	×70
×42	×62.5
×38	×56.5
×34	×50.5
WT12×31	WT12×46
×27.5	×41

Table 17–5 (continued)
SI Equivalents of Standard U.S. Shape Profiles
WT Shapes

Shape	SI Equivalent
in. × lb/ft	mm × kg/m
WT10.5×100.5	WT265×150
×91	×136
×83	×124
×73.5	×109.5
×66	×98
×61	×91
×55.5	×82.5
×50.5	×75
WT10.5×46.5	WT265×69
×41.5	×61.5
×36.5	×54.5
×34	×50.5
×31	×46
×27.5	×41
×24	×36
WT10.5×28.5	WT265×42.5
×25	×37
×22	×33
WT9×155.5	WT230×232
×141.5	×210.5
×129	×192
×117	×174.5
×105.5	×157.5
×96	×143
×87.5	×130
×79	×117.5
×71.5	×106.5
×65	×96.5
×59.5	×88.5
×53	×79
×48.5	×72
×43	×64
×38	×56.5
WT9×35.5	WT230×53
×32.5	×48.5
×30	×44.5
×27.5	×41
×25	×37
WT9×23	WT230×34
×20	×30
×17.5	×26

Shape	SI Equivalent
in. × lb/ft	mm × kg/m
WT8×50	WT205×74.5
×44.5	×66
×38.5	×57
×33.5	×50
WT8×28.5	WT205×42.5
×25	×37.5
×22.5	×33.5
×20	×30
×18	×26.5
WT8×15.5	WT205×23.05
×13	×19.4
WT7×365	WT180×543
×332.5	×495
×302.5	×450
×275	×409
×250	×372
×227.5	×338.5
×213	×317
×199	×296
×185	×275.5
×171	×254.5
×155.5	×231.5
×141.5	×210.5
×128.5	×191
×116.5	×173.5
×105.5	×157
×96.5	×143.5
×88	×131
×79.5	×118.5
×72.5	×108
WT7×66	WT180×98
×60	×89.5
×54.5	×81
×49.5	×73.5
×45	×67
WT7×41	WT180×61
×37	×55
×34	×50.5
×30.5	×45.5

Shape	SI Equivalent
in. × lb/ft	mm × kg/m
WT7×26.5	WT180×39.5
×24	×36
×21.5	×32
WT7×19	WT180×29
×17	×25.5
×15	×22.3
WT7×13	WT180×19.5
×11	×16.45
WT6×168	WT155×250
×152.5	×227
×139.5	×207.5
×126	×187.5
×115	×171
×105	×156.5
×95	×141.5
×85	×126.5
×76	×113
×68	×101
×60	×89.5
×53	×79
×48	×71.5
×43.5	×64.5
×39.5	×58.5
×36	×53.5
×32.5	×48.5
WT6×29	WT6×43
×26.5	×39.5
WT6×25	×37
×22.5	×33.5
×20	×30
WT6×17.5	WT6×26
×15	×22.25
×13	×19.35
WT6×11	WT6×16.35
×9.5	×14.15
×8	×11.9
×7	×10.5

Table 17–5 (continued)
SI Equivalents of Standard U.S. Shape Profiles
WT Shapes

Shape	SI Equivalent	Shape	SI Equivalent	Shape	SI Equivalent
in. × lb/ft	mm × kg/m	in. × lb/ft	mm × kg/m	in. × lb/ft	mm × kg/m
WT5×56	WT125×83.5	WT5×9.5	WT125×14.2	WT4×7.5	WT100×11.25
×50	×74.5	×8.5	×12.65	×6.5	×9.65
×44	×65.5	×7.5	×11.15	×5	×7.5
×38.5	×57.5	×6	×8.95	WT3×12.5	WT75×18.55
×34	×50.5	WT4×33.5	WT100×50	×10	×14.9
×30	×44.5	×29	×43	×7.5	×11.25
×27	×40	×24	×35.5	WT3×8	WT75×12
×24.5	×36.5	×20	×29.5	×6	×9
WT5×22.5	WT125×33.5	×17.5	×26	×4.5	×6.75
×19.5	×29	×15.5	×23.05	×4.25	×6.5
×16.5	×24.55	WT4×14	WT100×20.85	WT2.5×9.5	WT65×14.05
WT5×15	WT125×22.4	×12	×17.95	×8	×11.9
×13	×19.25	WT4×10.5	WT100×15.65	WT2×6.5	WT50×9.65
×11	×16.35	×9	×13.3		

Table 17–6
SI Equivalents of Standard U.S. Shape Profiles
MT and ST Shapes

Shape	SI Equivalent
in. × lb/ft	**mm × kg/m**
MT6.25×6.2	MT159×9.70
×5.8	×8.65
MT6×5.9	MT155×8.80
MT6×5.4	MT155×8.05
MT6×5	MT125×7.45
MT5×4.5	MT125×6.70
5×4	×5.95
MT5×3.75	MT125×5.60
MT4×3.25	MT100×4.85
×3.1	×4.25
MT3×2.2	MT75×3.3
×1.85	×2.75
MT2.5×9.45	MT65×14.1
MT2×3	MT50×4.45
×2.04	×3.05
×1.725	×2.55
×1.6	×2.4
MT1.5×1.45	MT37.5×2.15

Shape	SI Equivalent
in. × lb/ft	**mm × kg/m**
ST12×60.5	ST305×90
×53	×79
ST12×50	ST305×75
×45	×67
×40	×60
ST10×48	ST254×72
×43	×64
ST10×37.5	ST254×56
×33	×49
ST9×35	ST230×52
×27.35	×41
ST7.5×25	ST190×37
×21.45	×32
ST6×25	ST152×37
×20.4	×30
ST6×17.5	ST152×26
×15.9	×24
ST5×17.5	ST127×26
×12.7	×19
ST4×11.5	ST102×17
×9.2	×14
ST3×8.6	ST76.2×13
×6.25	×9.3
ST2.5×5	ST63.5×7.5
ST2×4.75	ST50.8×7.1
×3.85	×5.7
ST1.5×3.75	ST38.1×5.6
×2.85	×4.25

Table 17–7
SI Equivalents of Standard U.S. Shape Profiles
Rectangular HSS

Shape	SI Equivalent	Shape	SI Equivalent
in. × in. × in.	mm × mm × mm	in. × in. × in.	mm × mm × mm
HSS20×12×5/8	HSS508×304.8×15.9	HSS14×6×5/8	HSS355.6×152.4×15.9
×1/2	×12.7	×1/2	×12.7
×3/8	×9.5	×3/8	×9.5
×5/16	×7.9	×5/16	×7.9
HSS20×8×5/8	HSS508×203.2×15.9	×1/4	×6.4
×1/2	×12.7	×3/16	×4.8
×3/8	×9.5	HSS14×4×5/8	HSS355.6×101.6×15.9
×5/16	×7.9	×1/2	×12.7
HSS20×4×1/2	HSS508×101.6×12.7	×3/8	×9.5
×3/8	×9.5	×5/16	×7.9
×5/16	×7.9	×1/4	×6.4
×1/4	×6.4	×3/16	×4.8
HSS18×6×5/8	HSS457.2×152.4×15.9	HSS12×10×1/2	HSS304.8×254×12.7
×1/2	×12.7	×3/8	×9.5
×3/8	×9.5	×5/16	×7.9
×5/16	×7.9	×1/4	×6.4
×1/4	×6.4	HSS12×8×5/8	HSS304.8×203.2×15.9
HSS16×12×5/8	HSS406.4×304.8×15.9	×1/2	×12.7
×1/2	×12.7	×3/8	×9.5
×3/8	×9.5	×5/16	×7.9
×5/16	×7.9	×1/4	×6.4
HSS16×8×5/8	HSS406.4×203.2×15.9	×3/16	×4.8
×1/2	×12.7	HSS12×6×5/8	HSS304.8×152.4×15.9
×3/8	×9.5	×1/2	×12.7
×5/16	×7.9	×3/8	×9.5
×1/4	×6.4	×5/16	×7.9
HSS16×4×5/8	HSS406.4×101.6×15.9	×1/4	×6.4
×1/2	×12.7	×3/16	×4.8
×3/8	×9.5	HSS12×4×5/8	HSS304.8×101.6×15.9
×5/16	×7.9	×1/2	×12.7
×1/4	×6.4	×3/8	×9.5
×3/16	×4.8	×5/16	×7.9
HSS14×10×5/8	HSS355.6×254×15.9	×1/4	×6.4
×1/2	×12.7	×3/16	×4.8
×3/8	×9.5	HSS12×3½×3/8	HSS304.8×88.9×9.5
×5/16	×7.9	×5/16	×7.9
×1/4	×6.4	HSS12×3×5/16	HSS304.8×76.2×7.9
		×1/4	×6.4
		×3/16	×4.8

Table 17-7 (continued)
SI Equivalents of Standard U.S. Shape Profiles
Rectangular HSS

Shape	SI Equivalent
in. × in. × in.	mm × mm × mm
HSS12×2×3/8	HSS304.8×50.8×7.9
×1/4	×6.4
×3/16	×4.8
HSS10×8×5/8	HSS254×203.2×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS10×6×5/8	HSS254×152.4×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS10×5×3/8	HSS254×127×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS10×4×5/8	HSS254×101.6×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS10×3 1/2×1/2	HSS254×88.9×4.8
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS10×3×3/8	HSS254×76.2×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS10×2×3/8	HSS254×50.8×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS9×7×5/8	HSS228.6×177.8×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS9×5×5/8	HSS228.6×127×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS9×3×1/2	HSS228.6×76.2×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS8×6×5/8	HSS203.2×152.4×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS8×4×5/8	HSS203.2×101.6×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS8×3×1/2	HSS203.2×76.2×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2

Table 17-7 (continued)
SI Equivalents of Standard U.S. Shape Profiles
Rectangular HSS

Shape	SI Equivalent	Shape	SI Equivalent
in. × in. × in.	mm × mm × mm	in. × in. × in.	mm × mm × mm
HSS8×2×3/8	HSS203.2×50.8×9.5	HSS6×3×1/2	HSS152.4×76.2×12.7
×5/16	×7.9	×3/8	×9.5
×1/4	×6.4	×5/16	×7.9
×3/16	×4.8	×1/4	×6.4
×1/8	×3.2	×3/16	×4.8
HSS7×5×1/2	HSS177.8×127×12.7	×1/8	×3.2
×3/8	×9.5	HSS6×2×3/8	HSS152.4×50.8×9.5
×5/16	×7.9	×5/16	×7.9
×1/4	×6.4	×1/4	×6.4
×3/16	×4.8	×3/16	×4.8
×1/8	×3.2	×1/8	×3.2
HSS7×4×1/2	HSS177.8×101.6×12.7	HSS5×4×1/2	HSS127×101.6×12.7
×3/8	×9.5	×3/8	×9.5
×5/16	×7.9	×5/16	×7.9
×1/4	×6.4	×1/4	×6.4
×3/16	×4.8	×3/16	×4.8
×1/8	×3.2	×1/8	×3.2
HSS7×3×1/2	HSS177.8×76.2×12.7	HSS5×3×1/2	HSS127×76.2×12.7
×3/8	×9.5	×3/8	×9.5
×5/16	×7.9	×5/16	×7.9
×1/4	×6.4	×1/4	×6.4
×3/16	×4.8	×3/16	×4.8
×1/8	×3.2	×1/8	×3.2
HSS7×2×1/4	HSS177.8×50.8×6.4	HSS5×2 1/2×1/4	HSS127×63.5×6.4
×3/16	×4.8	×3/16	×4.8
×1/8	×3.2	×1/8	×3.2
HSS6×5×1/2	HSS152.4×127×12.7	HSS5×2×3/8	HSS127×50.8×9.5
×3/8	×9.5	×5/16	×7.9
×5/16	×7.9	×1/4	×6.4
×1/4	×6.4	×3/16	×4.8
×3/16	×4.8	×1/8	×3.2
×1/8	×3.2	HSS4×3×3/8	HSS101.6×76.2×9.5
HSS6×4×1/2	HSS152.4×101.6×12.7	×5/16	×7.9
×3/8	×9.5	×1/4	×6.4
×5/16	×7.9	×3/16	×4.8
×1/4	×6.4	×1/8	×3.2
×3/16	×4.8		
×1/8	×3.2		

Table 17–7 (continued)
SI Equivalents of Standard U.S. Shape Profiles
Rectangular HSS

Shape	SI Equivalent
in. × in. × in.	mm × mm × mm
HSS4×2½×3/8	HSS101.6×63.5×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS4×2×3/8	HSS101.6×50.8×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS3½×2½×3/8	HSS88.9×63.5×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS3½×2×1/4	HSS88.9×50.8×6.4
×3/16	×4.8
×1/8	×3.2
HSS3½×1½×1/4	HSS88.9×38.1×6.4
×3/16	×4.8
×1/8	×3.2
HSS3×2½×5/16	HSS76.2×63.5×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS3×2×5/16	HSS76.2×50.8×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS3×1½×1/4	HSS76.2×38.1×6.4
×3/16	×4.8
×1/8	×3.2
HSS3×1×3/16	HSS76.2×25.4×4.8
×1/8	×3.2
HSS2½×2×1/4	HSS63.5×50.8×6.4
×3/16	×4.8
×1/8	×3.2
HSS2½×1½×1/4	HSS63.5×38.1×6.4
×3/16	×4.8
×1/8	×3.2
HSS2½×1×3/16	HSS63.5×25.4×4.8
×1/8	×3.2
HSS2¼×2×3/16	HSS57.2×50.8×4.8
×1/8	×3.2
HSS2×1½×3/16	HSS50.8×38.1×4.8
×1/8	×3.2
HSS2×1×3/16	HSS50.8×25.4×4.8
×1/8	×3.2

Table 17–8
SI Equivalents of Standard U.S. Shape Profiles
Square HSS

Shape	SI Equivalent*
in. × in.	mm × mm
HSS16×16×5/8	HSS406.4×406.4×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
HSS14×14×5/8	HSS355.6×355.6×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
HSS12×12×5/8	HSS304.8×304.8×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS10×10×5/8	HSS254×254×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS9×9×5/8	HSS228.6×228.6×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS8×8×5/8	HSS203.2×203.2×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2

Shape	SI Equivalent*
in. × in.	mm × mm
HSS7×7×5/8	HSS177.8×177.8×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS6×6×5/8	HSS152.4×152.4×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS5 1/2×5 1/2×3/8	HSS139.7×139.7×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS5×5×1/2	HSS127×127×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS4 1/2×4 1/2×1/2	HSS114.3×114.3×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS4×4×1/2	HSS101.6×101.6×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2

Table 17–8 (continued)
SI Equivalents of Standard U.S. Shape Profiles
Square HSS

Shape	SI Equivalent	Shape	SI Equivalent
in. × in.	mm × mm	in. × in.	mm × mm
HSS3$^1/_2$×3$^1/_2$×$^3/_8$	HSS88.9×88.9×9.5	HSS2$^1/_2$×2$^1/_2$×$^5/_{16}$	HSS63.5×63.5×7.9
×$^5/_{16}$	×7.9	×$^1/_4$	×6.4
×$^1/_4$	×6.4	×$^3/_{16}$	×4.8
×$^3/_{16}$	×4.8	×$^1/_8$	×3.2
×$^1/_8$	×3.2	HSS2$^1/_4$×2$^1/_4$×$^1/_4$	HSS57.2×57.2×6.4
HSS3×3×$^3/_8$	HSS76.2×76.2×9.5	×$^3/_{16}$	×4.8
×$^5/_{16}$	×7.9	×$^1/_8$	×3.2
×$^1/_4$	×6.4	HSS2×2×$^1/_4$	HSS50.8×50.8×6.4
×$^3/_{16}$	×4.8	×$^3/_{16}$	×4.8
×$^1/_8$	×3.2	×$^1/_8$	×3.2

Table 17–9
SI Equivalents of Standard U.S. Shape Profiles
Round HSS and Pipe

Shape	SI Equivalent
in. × in.	mm × mm
HSS20.000×0.500	HSS508×12.7
×0.375	×9.5
HSS18.000×0.500	HSS457.2×12.7
×0.375	×9.5
HSS16.000×0.625	HSS406.4×15.9
×0.500	×12.7
×0.438	×11.1
×0.375	×9.5
×0.312	×7.9
×0.250	×6.4
HSS14.000×0.625	HSS355.6×15.9
×0.500	×12.7
×0.375	×9.5
×0.312	×7.9
×0.250	×6.4
HSS12.750×0.500	HSS323.9×12.7
×0.375	×9.5
×0.250	×6.4
HSS10.750×0.500	HSS273.1×12.7
×0.375	×9.5
×0.250	×6.4
HSS10.000×0.625	HSS254×15.9
×0.500	×12.7
×0.375	×9.5
×0.312	×7.9
×0.250	×6.4
×0.188	×4.8
HSS9.625×0.500	HSS244.5×12.7
×0.375	×9.5
×0.312	×7.9
×0.250	×6.4
×0.188	×4.8
HSS8.625×0.625	HSS219.1×15.9
×0.500	×12.7
×0.375	×9.5
×0.322	×8.2
×0.250	×6.4
×0.188	×4.8
HSS7.625×0.375	HSS193.7×9.5
×0.328	×8.3
HSS7.500×0.500	HSS190.5×12.7
×0.375	×9.5
×0.312	×7.9
×0.250	×6.4
×0.188	×4.8
HSS7.000×0.500	HSS177.8×12.7
×0.375	×9.5
×0.312	×7.9
×0.250	×6.4
×0.188	×4.8
×0.125	×3.2
HSS6.875×0.500	HSS174.6×12.7
×0.375	×9.5
×0.312	×7.9
×0.250	×6.4
×0.188	×4.8
HSS6.625×0.500	HSS168.3×12.7
×0.432	×11
×0.375	×9.5
×0.312	×7.9
×0.280	×7.1
×0.250	×6.4
×0.188	×4.8
×0.125	×3.2
HSS6.000×0.500	HSS152.4×12.7
×0.375	×9.5
×0.312	×7.9
×0.280	×7.1
×0.250	×6.4
×0.188	×4.8
×0.125	×3.2
HSS5.563×0.500	HSS141.3×12.7
×0.375	×9.5
×0.258	×6.6
×0.188	×4.8
×0.134	×3.4
HSS5.500×0.500	HSS139.7×12.7
×0.375	×9.5
×0.258	×6.6
HSS5.000×0.500	HSS127×12.7
×0.375	×9.5
×0.312	×7.9
×0.258	×6.6
×0.250	×6.4
×0.188	×4.8
×0.125	×3.2
HSS4.500×0.375	HSS114.3×9.5
×0.337	×8.6
×0.237	×6.0
×0.188	×4.8
×0.125	×3.2

Table 17–9 (continued)
SI Equivalents of Standard U.S. Shape Profiles
Round HSS and Pipe

Shape in. × in.	SI Equivalent mm × mm
HSS4.000×0.313	HSS101.6×8.0
×0.250	×6.4
×0.237	×6.0
×0.226	×5.7
×0.220	×5.6
×0.188	×4.8
×0.125	×3.2
HSS3.500×0.313	HSS88.9×8
×0.300	×7.6
×0.250	×6.4
×0.216	×5.5
×0.203	×5.2
×0.188	×4.8
×0.125	×3.2
HSS3.000×0.250	HSS76.2×6.4
×0.216	×5.5
×0.203	×5.2
×0.188	×4.8
×0.152	×3.9
×0.134	×3.4
×0.125	×3.2
HSS2.875×0.250	HSS73×6.4
×0.203	×5.2
×0.188	×4.8
×0.125	×3.2
HSS2.500×0.250	HSS63.5×6.4
×0.188	×4.8
×0.125	×3.2
HSS2.375×0.250	HSS60.3×6.4
×0.218	×5.5
×0.188	×4.8
×0.154	×3.9
×0.125	×3.2
HSS1.900×0.188	HSS48.3×4.8
×0.145	×3.7
×0.120	×3.0
HSS1.660×0.140	HSS42.2×3.6

Shape in. × in.	SI Equivalent mm × mm
PIPE ½ STD	PIPE 13 STD
PIPE ¾ STD	PIPE 19 STD
PIPE 1 STD	PIPE 25 STD
PIPE 1¼ STD	PIPE 32 STD
PIPE 1½ STD	PIPE 38 STD
PIPE 2 STD	PIPE 51 STD
PIPE 2½ STD	PIPE 64 STD
PIPE 3 STD	PIPE 75 STD
PIPE 3½ STD	PIPE 89 STD
PIPE 4 STD	PIPE 102 STD
PIPE 5 STD	PIPE 127 STD
PIPE 6 STD	PIPE 152 STD
PIPE 8 STD	PIPE 203 STD
PIPE 10 STD	PIPE 254 STD
PIPE 12 STD	PIPE 310 STD
PIPE ½ XS	PIPE 13 XS
PIPE ¾ XS	PIPE 19 XS
PIPE 1 XS	PIPE 25 XS
PIPE 1¼ XS	PIPE 32 XS
PIPE 1½ XS	PIPE 38 XS
PIPE 2 XS	PIPE 51 XS
PIPE 2½ XS	PIPE 64 XS
PIPE 3 XS	PIPE 75 XS
PIPE 3½ XS	PIPE 89 XS
PIPE 4 XS	PIPE 102 XS
PIPE 5 XS	PIPE 127 XS
PIPE 6 XS	PIPE 152 XS
PIPE 8 XS	PIPE 203 XS
PIPE 10 XS	PIPE 254 XS
PIPE 12 XS	PIPE 310 XS
PIPE 2 XXS	PIPE 51 XXS
PIPE 2½ XXS	PIPE 64 XXS
PIPE 3 XXS	PIPE 75 XXS
PIPE 4 XXS	PIPE 102 XXS
PIPE 5 XXS	PIPE 127 XXS
PIPE 6 XXS	PIPE 152 XXS
PIPE 8 XXS	PIPE 203 XXS

Table 17–10
Wire and Sheet Metal Gages
Equivalent thickness in decimals of an inch

Gage No.	U.S. Standard Gage for Uncoated Hot-& Cold-Rolled Sheets[b]	Galvanized Sheet Gage for Hot-Dipped Zinc-Coated Sheets[b]	USA Steel Wire Gage	Gage No.	U.S. Standard Gage for Uncoated Hot-& Cold-Rolled Sheets[b]	Galvanized Sheet Gage for Hot-Dipped Zinc-Coated Sheets[b]	USA Steel Wire Gage
7/0	–	–	.490	13	.0897	.0934	.092[a]
6/0	–	–	.462[a]	14	.0747	.0785	.080
5/0	–	–	.430[a]	15	.0673	.0710	.072
4/0	–	–	.394[a]	16	.0593	.0635	.062[a]
3/0	–	–	.362[a]	17	.0538	.0575	.054
2/0	–	–	.331	18	.0478	.0516	.048[a]
1/0	–	–	.306	19	.0418	.0456	.041
1	–	–	.283	20	.0359	.0396	.035[a]
2	–	–	.262[a]	21	.0329	.0366	–
3	.2391	–	.244[a]	22	.0299	.0336	–
4	.2242	–	.225[a]	23	.0269	.0306	–
5	.2092	–	.207	24	.0239	.0276	–
6	.1943	–	.192	25	.0209	.0247	–
7	.1793	–	.177	26	.0179	.0217	–
8	.1644	.1681	.162	27	.0164	.0202	–
9	.1495	.1532	.148[a]	28	.0149	.0187	–
10	.1345	.1382	.135	29	–	.0172	–
11	.1196	.1233	.120[a]	30	–	.0157	–
12	.1046	.1084	.106[a]				

[a]Rounded value. The steel wire page has been taken from ASTM A510 "General Requirements for Wire Rods and Coarse Round Wire, Carbon Steel." Sizes originally quoted to four decimal equivalent places have been rounded to three decimal places in accordance with rounding procedures of ASTM "Recommended Practice" E29.

[b]The equivalent thicknesses are for information only. The product is commonly specified to decimal thickness (mils), not to gage number.

Table 17–11
Coefficients of Expansion

The coefficient of linear expansion (ε) is the change in length, per unit of length, for a change of one degree of temperature. The coefficient of surface expansion is approximately two times the linear coefficient, and the coefficient of volume expansion, for solids, is approximately three times the linear coefficient.

A bar, free to move, will increase in length with an increase in temperature and will decrease in length with a decrease in temperature. The change in length will be εtl, where ε is the coefficient of linear expansion, t the change in temperature and l the length. If the ends of a bar are fixed, a change in temperature (t) will cause a change in the unit stress of $E\varepsilon t$, and in force of $AE\varepsilon t$, where A is the cross-sectional area of the bar and E the modulus of elasticity.

The following table gives the coefficient of linear expansion for 100°, or 100 times the value indicated above.

Example: A piece of medium steel is exactly 40 ft long at 60°F. Find the length at 90°F assuming the ends are free to move.

$$\text{change of length} = \varepsilon tl = \frac{.00065 \times 30 \times 40}{100} = .0078 \text{ ft}$$

The length at 90°F is 40.0078 ft

Example: A piece of medium carbon steel is exactly 40 ft long and the ends are fixed. If the temperature increases 30°F, what is the resulting change in the unit stress?

$$\text{change in unit stress} = E\varepsilon t = \frac{29{,}000 \times .00065 \times 30}{100} = 5.7 \text{ ksi}$$

COEFFICIENTS OF EXPANSION FOR 100 DEGREES = 100ε

Materials	Linear Expansion		Materials	Linear Expansion	
	Celcius	Fahren-heit		Celcius	Fahren-heit
METALS AND ALLOYS			STONE AND MASONRY		
Aluminum, wrought	.00231	.00128	Ashlar masonry	.00063	.00035
Brass	.00188	.00104	Brick masonry	.00061	.00034
Bronze	.00181	.00101	Cement, portland	.00126	.00070
Copper	.00168	.00093	Concrete	.00099	.00055
Iron, cast, gray	.00106	.00059	Granite	.00080	.00044
Iron, wrought	.00120	.00067	Limestone	.00076	.00042
Iron, wire	.00124	.00069	Marble	.00081	.00045
Lead	.00286	.00159	Plaster	.00166	.00092
Magnesium, various alloys	.0029	.0016	Rubble masonry	.00063	.00035
Nickel	.00126	.00070	Sandstone	.00097	.00054
Steel, mild	.00117	.00065	Slate	.00080	.00044
Steel, stainless, 18-8	.00178	.00099			
Zinc, rolled	.00311	.00173			
TIMBER			TIMBER		
Fir (parallel to fiber)	.00037	.00021	Fir (perpendicular to fiber)	.0058	.0032
Maple (parallel to fiber)	.00064	.00036	Maple (perpendicular to fiber)	.0048	.0027
Oak (parallel to fiber)	.00049	.00027	Oak (perpendicular to fiber)	.0054	.0030
Pine (parallel to fiber)	.00054	.00030	Pine (perpendicular to fiber)	.0034	.0019

EXPANSION OF WATER

Maximum Density = 1

°C	Volume	°C	Volume	°C	Volume	°C	Volume	°C	Volume	°C	Volume
0	1.000126	10	1.000257	30	1.004234	50	1.011877	70	1.022384	90	1.035829
4	1.000000	20	1.001732	40	1.007627	60	1.016954	80	1.029003	100	1.043116

Table 17–12
Densities of Common Materials

Substance	Weight lb per ft^3	Substance	Weight lb per ft^3
ASHLAR, MASONRY		River mud	90.0
Granite, syenite, gneiss	143 – 187	Soil	70.0
Limestone, marble	143 – 174	Stone riprap	65.0
Sandstone, bluestone	131 – 150		
		MINERALS	
MORTAR RUBBLE MASONRY		Asbestos	131 – 174
Granite, syenite, gneiss	137 – 174	Barytes	280
Limestone, marble	137 – 162	Basalt	168 – 199
Sandstone, bluestone	125 – 137	Bauxite	159
		Borax	106 – 112
DRY RUBBLE MASONRY		Chalk	112 – 162
Granite, syenite, gneiss	118 – 143	Clay, marl	112 – 162
Limestone, marble	118 – 131	Dolomite	181
Sandstone, bluestone	112 – 118	Feldspar, orthoclase	156 – 162
		Gneiss, serpentine	150 – 168
BRICK MASONRY		Granite, syenite	156 – 193
Pressed brick	137 – 143	Greenstone, trap	174 – 199
Common brick	112 – 125	Gypsum, alabaster	143 – 174
Soft brick	93.5 – 106	Hornblende	187
		Limestone, marble	156 – 174
CONCRETE MASONRY		Magnesite	187
Cement, stone, sand	137 – 150	Phosphate rock, apatite	199
Cement, slag, etc.	118 – 143	Porphyry	162 – 181
Cement, cinder, etc.	93.5 – 106	Pumice, natural	23.1 – 56.1
		Quartz, flint	156 – 174
VARIOUS BUILDING MATERIALS		Sandstone, bluestone	137 – 156
Ashes, cinders	40.0 – 45.0	Shale, slate	168 – 181
Cement, portland, loose	90.0	Soapstone, talc	162 – 174
Cement, portland, set	168 – 199		
Lime, gypsum, loose	53.0 – 64.0	**STONE, QUARRIED, PILED**	
Mortar, set	87.2 – 118	Basalt, granite, gneiss	96.0
Slags, bank slag	67.0 – 72.0	Limestone, marble, quartz	95.0
Slags, bank screenings	98 – 117	Sandstone	82.0
Slags, machine slag	96.0	Shale	92.0
Slag, slag sand	49.0 – 55.0	Greenstone, hornblende	107
EARTH, ETC., EXCAVATED		**BITUMINOUS SUBSTANCES**	
Clay, dry	63.0	Asphaltum	68.5 – 93.5
Clay, damp, plastic	110	Coal, anthracite	87.2 – 106
Clay and gravel, dry	100	Coal, bituminous	74.8 – 93.5
Earth, dry, loose	76.0	Coal, lignite	68.5 – 87.2
Earth, dry, packed	95.0	Coal, peat, turf, dry	40.5 – 53
Earth, moist, loose	78.0	Coal, charcoal, pine	17.4 – 27.4
Earth, moist, packed	96.0	Coal, charcoal, oak	29.3 – 35.5
Earth, mud, flowing	108	Coal, coke	62.3 – 87.2
Earth, mud, packed	115	Graphite	118 – 143
Riprap, limestone	80.0 – 85.0	Paraffine	54.2 – 56.7
Riprap, sandstone	90.0	Petroleum	54.2
Riprap, shale	105	Petroleum, refined	49.2 – 51.1
Sand, gravel, dry, loose	90.0 – 105	Petroleum, benzine	45.5 – 46.7
Sand, gravel, dry, packed	100 – 120	Petroleum, gasoline	41.1 – 43
Sand, gravel, wet	118 – 120	Pitch	66.7 – 71.6
		Tar, bituminous	74.8
EXCAVATIONS IN WATER			
Sand or gravel	60.0	**COAL AND COKE, PILED**	
Sand or gravel and clay	65.0	Coal, anthracite	47.0 – 58.0
Clay	80.0	Coal, bituminous, lignite	40.0 – 54.0

Table 17–12 (continued)
Densities of Common Materials

Substance	Weight lb per ft^3	Substance	Weight lb per ft^3
Coal, peat, turf	20.0 – 26.0	Starch	95.3
Coal charcoal	10.0 – 14.0	Sulphur	120 – 129
Coal coke	23.0 – 32.0	Wool	82.2
METALS, ALLOYS, ORES		**TIMBER, U.S. SEASONED**	
Aluminum, cast, hammered	159 – 171	Moisture content by weight:	
Brass, cast, rolled	523 – 542	Seasoned timber 15 to 20%	
Bronze, 7.9 to 14% Sn	461 – 554	Green timber up to 50%	
Bronze, aluminum	480	Ash, white, red	38.6 – 40.5
Copper, cast, rolled	548 – 561	Cedar, white, red	19.9 – 23.7
Copper ore, pyrites	255 – 268	Chestnut	41.1
Gold, cast, hammered	1200–1210	Cypress	29.9
Iron, cast, pig	449	Fir, Douglas spruce	31.8
Iron, wrought	473 – 492	Fir, eastern	24.9
Iron, speigel–eisen	467	Elm, white	44.9
Iron, ferro–silicon	417 – 455	Hemlock	26.2 – 32.4
Iron ore, hematite	324	Hickory	46.1 – 52.3
Iron ore, hematite in bank	160 – 180	Locust	45.5
Iron ore, hematite loose	130 – 160	Maple, hard	42.4
Iron ore, limonite	224 – 249	Maple, white	33.0
Iron ore, magnetite	305 – 324	Oak, chestnut	53.6
Iron slag	156 – 187	Oak, live	59.2
Lead	710	Oak, red, black	40.5
Lead ore, galena	455 – 473	Oak, white	46.1
Magnesium, alloys	108 – 114	Pine, Oregon	31.8
Manganese	449 – 498	Pine, red	29.9
Manganese, ore, pyrolusite	231 – 287	Pine, white	25.5
Mercury	847	Pine, yellow, long–leaf	43.6
Monel Metal	548 – 561	Pine, yellow, short–leaf	38.0
Nickel	554 – 573	Poplar	29.9
Platinum, cast, hammered	1310 – 1340	Redwood, California	26.2
Silver, cast, hammered	648 – 668	Spruce, white, black	24.9 – 28.7
Steel, rolled	490	Walnut, black	38.0
Tin, cast, hammered	449 – 467	Walnut, white	25.5
Tin ore, cassiterite	399 – 436		
Zinc, cast, rolled	430 – 449	**VARIOUS LIQUIDS**	
Zinc, ore, blende	243 – 262	Alcohol, 100%	49.2
		Acids, muriatic 40%	74.8
VARIOUS SOLIDS		Acids, nitric 91%	93.5
Cereals, oats, bulk	32.0	Acids, sulphuric 87%	112
Cereals, barley, bulk	39.0	Lye, soda 66%	106
Cereals, corn, rye, bulk	48.0	Oils, vegetable	56.7 – 58.6
Cereals, wheat, bulk	48.0	Oils, mineral, lubricants	56.1 – 57.9
Hay and Straw, bales	20.0	Water, 4°C max. density	62.3
Cotton, Flax, Hemp	91.6 – 93.5	Water, 100°C	59.7
Fats	56.1 – 60.4	Water, ice	54.8 – 57.3
Flour, loose	24.9 – 31.2	Water, sea water	63.5 – 64.2
Flour, pressed	43.6 – 49.8		
Glass, common	150 – 162	**GASES**	
Glass, plate or crown	153 – 169	Air, 0°C 760 mm	0.0871
Glass, crystal	181 – 187	Ammonia	0.0478
Leather	53.6 – 63.5	Carbon dioxide	0.123
Paper	43.6 – 71.6	Carbon monoxide	0.078
Potatoes, piled	42.0	Gas, illuminating	0.028–0.036
Rubber, caoutchouc	57.3 – 59.8	Gas, natural	0.038–0.039
Rubber goods	62.3 – 125	Hydrogen	0.00559
Sald, granulated, piled	48.0	Nitrogen	0.0784
Saltpeter	67.0	Oxygen	0.0892

Table 17–13
Weights of Building Materials

Materials	Weight lb per sq ft	Materials	Weight lb per sq ft
CEILINGS		**PARTITIONS**	
Channel suspended system	1	Clay Tile	
Lathing and plastering	See Partitions	3 in.	17
Acoustical fiber tile	1	4 in.	18
		6 in.	28
		8 in.	34
		10 in.	40
FLOORS		Gypsum Block	
Steel Deck	See Manufacturer	2 in.	9½
		3 in.	10½
Concrete-Reinforced 1 in.		4 in.	12½
Stone	12½	5 in.	14
Slag	11½	6 in.	18½
Lightweight	6 to 10	Wood Studs 2×4	
		12–16 in. o.c.	2
Concrete-Plain 1 in.		Steel partitions	4
Stone	12	Plaster 1 inch	
Slag	11	Cement	10
Lightweight	3 to 9	Gypsum	5
		Lathing	
Fills 1 inch		Metal	½
Gypsum	6	Gypsum Board ½-in.	2
Sand	8		
Cinders	4		
Finishes			
Terrazzo 1 in.	13		
Ceramic or Quarry Tile ¾-in.	10		
Linoleum ¼-in.	1	**WALLS**	
Mastic ¾-in.	9	Brick	
Hardwood ⅞-in.	4	4 in.	40
Softwood ¾-in.	2½	8 in.	80
		12 in.	120
		Hollow Concrete Block	
		(Heavy Aggregate)	
ROOFS		4 in.	30
Copper or tin	1	6 in.	43
Corrugated steel	See Manufactuer	8 in.	55
3-ply ready roofing	1	12½-in.	80
3-ply felt and gravel	5½	Hollow Concrete Block	
5-ply felt and gravel	6	(Light Aggregate)	
		4 in.	21
Shingles		6 in.	30
Wood	2	8 in.	38
Asphalt	3	12 in.	55
Clay tile	9 to 14	Clay tile (Load Bearing)	
Slate ¼	10	4 in.	25
		6 in.	30
Sheathing		8 in.	33
Wood ¾-in.	3	12 in.	45
Gypsum 1 in.	4	Stone 4 in.	55
		Glass Block 4 in.	18
Insulation 1 in.		Window, Glass, Frame, & Sash	8
Loose	½	Curtain Walls	See Manufacturer
Poured	2	Structural Glass 1 in.	15
Rigid	1½	Corrugated Cement Asbestos ¼-in.	3

For weights of other materials used in building construction, see Table 17-12.

Table 17–14
Weights and Measures
United States System

LINEAR MEASURE

Inches	Feet	Yards	Rods	Furlongs	Miles
1.0 =	.08333 =	.02778 =	.0050505 =	.00012626 =	.00001578
12.0 =	1.0 =	.33333 =	.0606061 =	.00151515 =	.00018939
36.0 =	3.0 =	1.0 =	.1818182 =	.00454545 =	.00056818
198.0 =	16.5 =	5.5 =	1.0 =	.025 =	.003125
7,920.0 =	660.0 =	220.0 =	40.0 =	1.0 =	.125
63,360.0 =	5,280.0 =	1,760.0 =	320.0 =	8.0 =	1.0

SQUARE AND LAND MEASURE

Sq. Inches	Square Feet	Square Yards	Square Rods	Acres	Sq. Miles
1.0 =	.006944 =	.000772			
144.0 =	1.0 =	.111111			
1,296.0 =	9.0 =	1.0 =	.03306 =	.000207	
39,204.0 =	272.25 =	30.25 =	1.0 =	.00625 =	.0000098
	43,560.0 =	4,840.0 =	160.0 =	1.0 =	.0015625
		3,097,600.0 =	102,400.0 =	640.0 =	1.0

AVOIRDUPOIS WEIGHTS

Grains	Drams	Ounces	Pounds	Tons
1.0 =	.03657 =	.002286 =	.000143 =	.0000000714
27.34375 =	1.0 =	.0625 =	.003906 =	.00000195
437.5 =	16.0 =	1.0 =	.0625 =	.00003125
7,000.0 =	256.0 =	16.0 =	1.0 =	.0005
14,000,000.0 =	512,000.0 =	32,000.0 =	2,000.0 =	1.0

DRY MEASURE

Pints	Quarts	Pecks	Cubic Feet	Bushels
1.0 =	.5 =	.0625 =	.01945 =	.01563
2.0 =	1.0 =	.125 =	.03891 =	.03125
16.0 =	8.0 =	1.0 =	.31112 =	.25
51.42627 =	25.71314 =	3.21414 =	1.0 =	.80354
64.0 =	32.0 =	4.0 =	1.2445 =	1.0

LIQUID MEASURE

Gills	Pints	Quarts	U.S. Gallons	Cubic Feet
1.0 =	.25 =	.125 =	.03125 =	.00418
4.0 =	1.0 =	.5 =	.125 =	.01671
8.0 =	2.0 =	1.0 =	.250 =	.03342
32.0 =	8.0 =	4.0 =	1.0 =	.1337
			7.48052 =	1.0

SI UNITS FOR STRUCTURAL STEEL DESIGN

Although there are seven metric base units in the SI system, only four are currently used by AISC in structural steel design. These base units are listed in Table 17-15.

Table 17–15. Base SI Units for Steel Design

Quantity	Unit	Symbol
Length	meter	m
mass	kilogram	kg
time	second	s
temperature	celcius	°C

Similarly, of the numerous decimal prefixes included in the SI system, only three are used in steel design; see Table 17-16.

Table 17–16. SI Prefixes for Steel Design

Prefix	Symbol	Order of Magnitude	Expression
mega	M	10^6	1,000,000 (one million)
kilo	k	10^3	1,000 (one thousand)
milli	m	10^{-3}	0.001 (one thousandth)

In addition, three derived units are applicable to the present conversion. They are shown in Table 17-17.

Table 17–17. Derived SI Units for Steel Design

Quantity	Name	Symbol	Expression
force	newton	N	$N = kg \times m/s^2$
stress	pascal	Pa	$Pa = N/m^2$
energy	joule	J	$J = N \times m$

Although specified in SI, the pascal is not universally accepted as the unit of stress. Because section properties are expressed in millimeters, it is more convenient to express stress in newtons per square millimeter ($1\ N/mm^2 = 1\ MPa$). This is the practice followed in recent international structural design standards. It should be noted that the joule, as the unit of energy, is used to express energy absorption requirements for impact tests. Moments are expressed in terms of N×m.

A summary of the conversion factors relating traditional U.S. units of measurement to the corresponding SI units is given in Table 17-18.

Table 17–18. Summary of SI Conversion Factors

Multiply	by:	to obtain:
inch (in.)	25.4	millimeters (mm)
foot (ft)	305	millimeters (mm)
pound-mass (lb)	0.454	kilogram (kg)
pound-force (lbf)	4.448	newton (N)
ksi	6.895	N/mm^2
ft-lbf	1.356	joule (J)
psf	47.88	N/m^2
plf	14.59	N / m

Note that fractions resulting from metric conversion should be rounded to whole millimeters. Common fractions of inches and their metric equivalents are in Table 17–19.

Table 17–19. SI Equivalents of Fractions of an Inch

Fraction, in.	Exact conversion, mm	Rounded to: (mm)
1/16	1.5875	2
1/8	3.175	3
3/16	4.7625	5
1/4	6.35	6
5/16	7.9375	8
3/8	9.525	10
7/16	11.1125	11
1/2	12.7	13
5/8	15.875	16
3/4	19.05	19
7/8	22.225	22
1	25.4	25

Bolt diameters are taken directly from the ASTM Specifications A325M and A490M rather than converting the diameters of SI bolts dimensioned in inches, since metric bolts are of different physical sizes. The metric bolt designations are in Table 17–20.

Table 17–20. SI Bolt Designation

Designation	Diameter, mm	Diameter, in.
M16	16	0.63
M20	20	0.79
M22	22	0.87
M24	24	0.94
M27	27	1.06
M30	30	1.18
M36	36	1.42

The yield strengths of structural steels are taken from the metric ASTM Specifications. It should be noted that the yield points are slightly different from the traditional values. See Table 17–21. The modulus of elasticity of steel E is taken as 200,000 N/mm^2. The shear modulus of elasticity of steel G is 77,000 N/mm^2.

Table 17–21. SI Steel Yield Stresses

ASTM Designation	Yield stress, N/mm^2	Yield stress, ksi
A36M	250	36.26
A572M Gr. 345 A588M	345	50.04
A852M	485	70.34
A514M	690	100.07

Table 17–22
Weights and Measures
International System of Units (SI)[a]
(Metric practice)

BASE UNITS

Quantity	*Unit*	*Symbol*
length	meter	m
mass	kilogram	kg
time	second	s
electric current	ampere	A
thermodynamic temperature	kelvin	K
amount of substance	mole	mol
luminous intensity	candela	cd

SUPPLEMENTARY UNITS

Quantity	*Unit*	*Symbol*
plane angle	radian	rad
solid angle	steradian	sr

DERIVED UNITS (WITH SPECIAL NAMES)

Quantity	*Unit*	*Symbol*	*Formula*
force	newton	N	kg-m/s^2
pressure, stress	pascal	Pa	N/m^2
energy, work, quantity of heat	joule	J	N-m
power	watt	W	J/s

DERIVED UNITS (WITHOUT SPECIAL NAMES)

Quantity	*Unit*	*Formula*
area	square meter	m^2
volume	cubic meter	m^3
velocity	meter per second	m/s
acceleration	meter per second squared	m/s^2
specific volume	cubic meter per kilogram	m^3/kg
density	kilogram per cubic meter	kg/m^3

SI PREFIXES

Multiplication Factor	*Prefix*	*Symbol*
1 000 000 000 000 000 000 = 10^{18}	exa	E
1 000 000 000 000 000 = 10^{15}	peta	P
1 000 000 000 000 = 10^{12}	tera	T
1 000 000 000 = 10^{9}	giga	G
1 000 000 = 10^{6}	mega	M
1 000 = 10^{3}	kilo	k
100 = 10^{2}	hecto[b]	h
10 = 10^{1}	deka[b]	da
0.1 = 10^{-1}	deci[b]	d
0.01 = 10^{-2}	centi[b]	c
0.001 = 10^{-3}	milli	m
0.000 001 = 10^{-6}	micro	μ
0.000 000 001 = 10^{-9}	nano	n
0.000 000 000 001 = 10^{-12}	pico	p
0.000 000 000 000 001 = 10^{-15}	femto	f
0.000 000 000 000 000 001 = 10^{-18}	atto	a

[a]Refer to ASTM E380 for more complete information on SI.
[b]Use is not recommended.

Table 17–23
SI Conversion Factors[a]

Quantity	Multiply	by	to obtain	
Length	inch	25.400	millimeter	mm
	foot	0.305	meter	m
	yard	0.914	meter	m
	mile (U.S. Statute)	1.609	kilometer	km
	millimeter	39.370×10^{-3}	inch	in
	meter	3.281	foot	ft
	meter	1.094	yard	yd
	kilometer	0.621	mile	mi
Area	square inch	0.645×10^{3}	square millimeter	mm^2
	square foot	0.093	square meter	m^2
	square yard	0.836	square meter	m^2
	square mile (U.S. Statute)	2.590	square kilometer	km^2
	acre	4.047×10^{3}	square meter	m^2
	acre	0.405	hectare	
	square millimeter	1.550×10^{-3}	square inch	in^2
	square meter	10.764	square foot	ft^2
	square meter	1.196	square yard	yd^2
	square kilometer	0.386	square mile	mi^2
	square meter	0.247×10^{-3}	acre	
	hectare	2.471	acre	
Volume	cubic inch	16.387×10^{3}	cubic millimeter	mm^3
	cubic foot	28.317×10^{-3}	cubic meter	m^3
	cubic yard	0.765	cubic meter	m^3
	gallon (U.S. liquid)	3.785	liter	*l*
	quart (U.S. liquid)	0.946	liter	*l*
	cubic millimeter	61.024×10^{-6}	cubic inch	in^3
	cubic meter	35.315	cubic foot	ft^3
	cubic meter	1.308	cubic yard	yd^3
	liter	0.264	gallon (U.S. liquid)	gal
	liter	1.057	quart (U.S. liquid)	qt
Mass	ounce (avoirdupois)	28.350	gram	g
	pound (avoirdupois)	0.454	kilogram	kg
	short ton	0.907×10^{3}	kilogram	kg
	gram	35.274×10^{-3}	ounce (avoirdupois)	oz av
	kilogram	2.205	pound (avoirdupois)	lb av
	kilogram	1.102×10^{-3}	short ton	

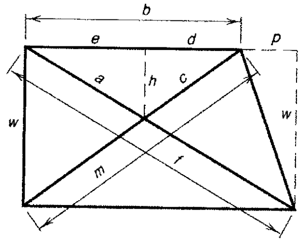
[a]Refer to ASTM E380 for more complete information on SI.
The conversion factors tabulated herein have been rounded.

Table 17–23 (continued)
SI Conversion Factors[a]

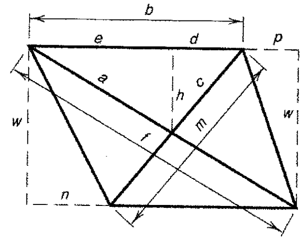
Quantity	Multiply	by	to obtain	
Force	[c]ounce-force	0.278	[c]newton	N
	[c]pound-force	4.448	[c]newton	N
	[c]newton	3.597	[c]ounce-force	
	[c]newton	0.225	[c]pound-force	lbf
Bending Moment	[c]pound-force-inch	0.113	[c]newton-meter	N-m
	[c]pound-force-foot	1.356	[c]newton-meter	N-m
	[c]newton-meter	8.851	[c]pound-force-inch	lbf-in
	[c]newton-meter	0.738	[c]pound-force-foot	lbf-ft
Pressure, Stress	[c]pound-force per square inch	6.895	[c]kilopascal	kPa
	[c]foot of water (39.2 F)	2.989	[c]kilopascal	kPa
	[c]inch of mercury (32 F)	3.386	[c]kilopascal	kPa
	[c]kilopascal	0.145	[c]pound-force per [c] square inch	lbf/in^2
	[c]kilopascal	0.335	[c]foot of water (39.2 F)	
	[c]kilopascal	0.295	[c]inch of mercury (32 F)	
Energy, Work, Heat	[c]foot-pound-force	1.356	[c]joule	J
	[b]British thermal unit	1.055×10^{3}	[c]joule	J
	[b]calorie	4.187	[c]joule	J
	[c]kilowatt hour	3.600×10^{6}	[c]joule	J
	[c]joule	0.738	[c]foot-pound-force	ft-lbf
	[c]joule	0.948×10^{-3}	[b]British thermal unit	Btu
	[c]joule	0.239	[b]calorie	
	[c]joule	0.278×10^{-6}	[c]kilowatt hour	kW-h
Power	[c]foot-pound-force/second	1.356	[c]watt	W
	[b]British thermal unit per hour	0.293	[c]watt	W
	[c]horsepower (550 ft lbf/s)	0.746	[c]kilowatt	kW
	[c]watt	0.738	[c]foot-pound-force/ [c] second	ft-lbf/s
	[c]watt	3.412	[b]British thermal unit [c] per hour	Btu/h
	kilowatt	1.341	[c]horsepower [c] (550 ft-lbf/s)	hp
Angle	[c]degree	17.453×10^{-3}	[c]radian	rad
	[c]radian	57.296	[c]degree	
Temperature	[c]degree Fahrenheit	t°C = (t°F – 32)/1.8	[c]degree Celsius	
	[c]degree Celsius	t°F = 1.8 × t°C + 32	[c]degree Fahrenheit	

[a]Refer to ASTM E380 for more complete information on SI.
[b]International Table.
[c]The conversion factors tabulated herein have been rounded.

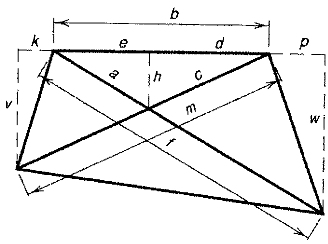
Table 17–24
Bracing Formulas



Given	To Find	Formula
bpw	f	$\sqrt{(b+p)^2+w^2}$
bw	m	$\sqrt{b^2+w^2}$
bp	d	$b^2 \div (2b+p)$
bp	e	$b(b+p) \div (2b+p)$
bfp	a	$bf \div (2b+p)$
bmp	c	$bm \div (2b+p)$
bpw	h	$bw \div (2b+p)$
afw	h	$aw \div f$
cmw	h	$cw \div m$



Given	To Find	Formula
bpw	f	$\sqrt{(b+p)^2+w^2}$
bnw	m	$\sqrt{(b-n)^2+w^2}$
bnp	d	$b(b-n) \div (2b+p-n)$
bnp	e	$b(b+p) \div (2b+p-n)$
bfnp	a	$bf \div (2b+p-n)$
bmnp	c	$bm \div (2b+p-n)$
bnpw	h	$bw \div (2b+p-n)$
afw	h	$aw \div f$
cmw	h	$cw \div m$



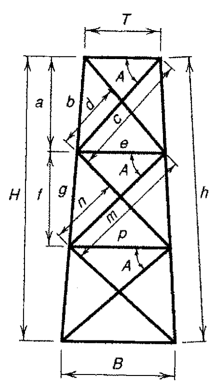
Given	To Find	Formula
bpw	f	$\sqrt{(b+p)^2+w^2}$
bkv	m	$\sqrt{(b+k)^2+v^2}$
bkpvw	d	$bw(b+k) \div [v(b+p)+w(b+k)]$
bkpvw	e	$bv(b+p) \div [v(b+p)+w(b+k)]$
bfkpvw	a	$fbv \div [v(b+p)+w(b+k)]$
bkmpvw	c	$bmw \div [v(b+p)+w(b+k)]$
bkpvw	h	$bvw \div [v(b+p)+w(b+k)]$
afw	h	$aw \div f$
cmv	h	$cv \div m$

PARALLEL BRACING

$k = (\log B - \log T) \div$ no. of panels. Constant k plus the logarithm of any line equals the log of the corresponding line in the next panel below.

$a = TH \div (T+e+p)$
$b = Th \div (T+e+p)$

$c = \sqrt{(1/2\,T + 1/2\,e)^2 + a^2}$
$d = ce \div (T+e)$

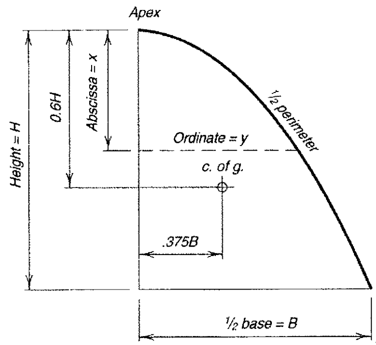


$\log e = k + \log T$
$\log f = k + \log a$
$\log g = k + \log b$
$\log m = k + \log c$
$\log n = k + \log d$
$\log p = k + \log e$

The above method can be used for any number of panels. In the formulas for "a" and "b" the sum in parenthesis, which in the case shown is $(T+e+p)$, is always composed of all the horizontal distances except the base.

Table 17–25
Properties of Parabola and Ellipse

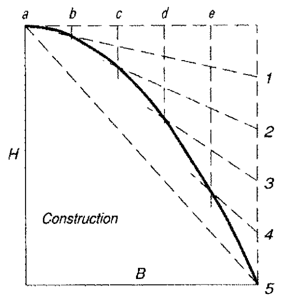
PARABOLA



Parameter $P = \frac{B^2}{H}$ Area = ⅔ HB

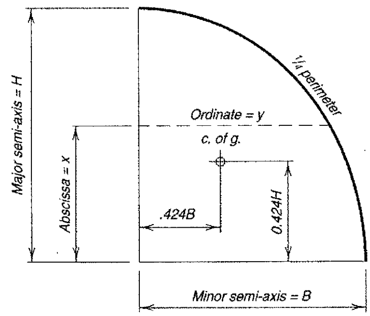
$x = \frac{y^2}{P}$

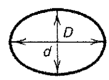
$y = \sqrt{xP}$



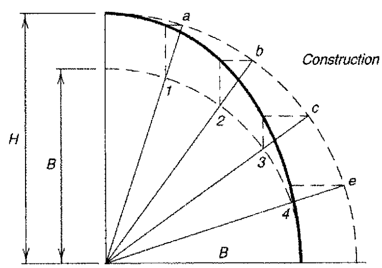
ELLIPSE

$(x^2 \div H^2) + (y^2 \div B^2) = 1$

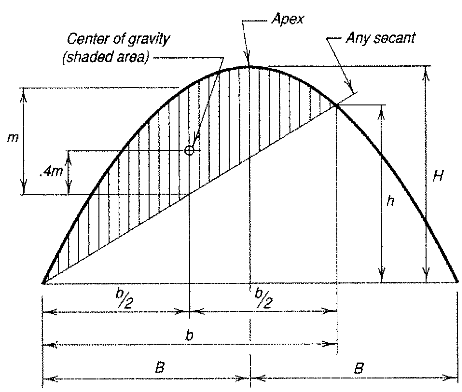




Area = .7854Dd

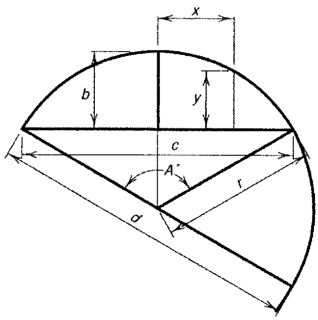


AREA BETWEEN PARABOLIC CURVE AND SECANT



Length b may vary from 0 to 2B

Table 17–26
Properties of the Circle



Circumference = 6.28378 r = 3.14159d
Diameter = 0.31831 circumference
Area = 3.14159r^2

$$\text{Arc } a = \frac{\pi r A^\circ}{180^\circ} = 0.017453 r A^\circ$$

$$\text{Angle } A^\circ = \frac{180^\circ a}{\pi r} = 57.29578\frac{a}{r}$$

$$\text{Angle } A^\circ = 2\sin^{-1}(c/2r)$$

$$\text{Angle } A^\circ = 4\tan^{-1}(2b/c)$$

$$\text{Radius } r = \frac{4b^2 + c^2}{8b}$$

$$\text{Chord } c = 2\sqrt{2br - b^2} = 2r\sin\frac{A}{2}$$

$$\text{Rise } b = r - \frac{1}{2}\sqrt{4r^2 - c^2} = \frac{c}{2}\tan\frac{A}{4}$$

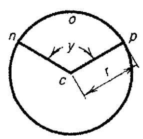
$$= 2r\sin^2\frac{A}{4} = r + y - \sqrt{r^2 - x^2}$$

$$y = b - r + \sqrt{r^2 - x^2}$$

$$x = \sqrt{r^2 - (r + y - b)^2}$$

Diameter of circle of equal periphery as square	= 1.27324 side of square
Side of square of equal periphery as circle	= 0.78540 diameter of circle
Diameter of circle circumscribed about square	= 1.41421 side of square
Side of square inscribed in circle	= 0.70711 diameter of circle

CIRCULAR SECTOR

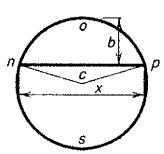


r = radius of circle y = angle ncp in degrees
Area of Sector $ncpo$ = ½ (length of arc $nop \times r$)

$$= \text{Area of Circle} \times \frac{y}{360}$$

$$= 0.0087266 \times r^2 \times y$$

CIRCULAR SEGMENT



r = radius of circle x = chord b = rise
Area of Segment nop = Area of Sector ncpo – Area of triangle ncp

$$= \frac{(\text{Length of arc } nop \times r) - x(r - b)}{2}$$

r = radius of circle y = angle ncp in degrees
Area of Sector $ncpo$ = ½ (length of arc $nop \times r$)

$$= \text{Area of Circle} \times \frac{y}{360}$$

$$= 0.0087266 \times r^2 \times y$$

Table 17–27
Properties of Geometric Sections

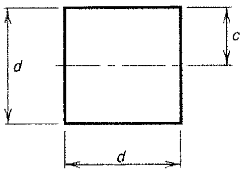
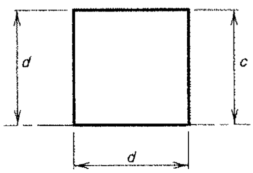
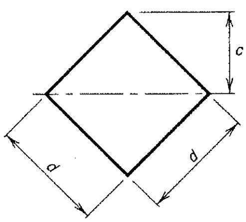
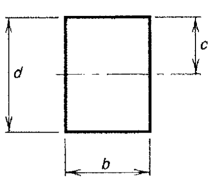
SQUARE *Axis of moments through center* 	$A = d^2$ $c = \dfrac{d}{2}$ $I = \dfrac{d^4}{12}$ $S = \dfrac{d^3}{6}$ $r = \dfrac{d}{\sqrt{12}} = .288675\ d$ $Z = \dfrac{d^3}{4}$
SQUARE *Axis of moments on base* 	$A = d^2$ $c = d$ $I = \dfrac{d^4}{3}$ $S = \dfrac{d^3}{3}$ $r = \dfrac{d}{\sqrt{3}} = .577350\ d$
SQUARE *Axis of moments on diagonal* 	$A = d^2$ $c = \dfrac{d}{\sqrt{2}} = .707107\ d$ $I = \dfrac{d^4}{12}$ $S = \dfrac{d^3}{6\sqrt{2}} = .117851\ d^3$ $r = \dfrac{d}{\sqrt{12}} = .288675\ d$ $Z = \dfrac{2c^3}{3} = \dfrac{d^3}{3\sqrt{2}} = .235702\ d^3$
RECTANGLE *Axis of moments through center* 	$A = bd$ $c = \dfrac{d}{2}$ $I = \dfrac{bd^3}{12}$ $S = \dfrac{bd^2}{6}$ $r = \dfrac{d}{\sqrt{12}} = .288675\ d$ $Z = \dfrac{bd^2}{4}$

Table 17–27 (continued)
Properties of Geometric Sections

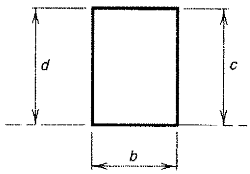
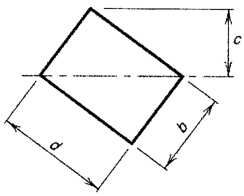
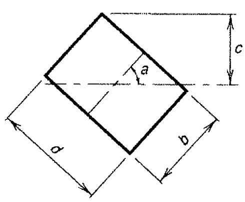
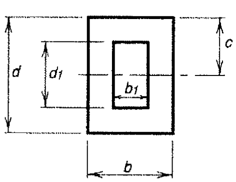
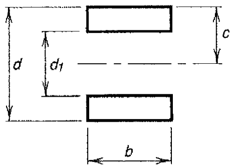
RECTANGLE *Axis of moments on base*  	$A = bd$ $c = d$ $I = \dfrac{bd^3}{3}$ $S = \dfrac{bd^2}{3}$ $r = \dfrac{d}{\sqrt{3}} = .577350\,d$
RECTANGLE *Axis of moments on diagonal*  	$A = bd$ $c = \dfrac{bd}{\sqrt{b^2 + d^2}}$ $I = \dfrac{b^3 d^3}{6(b^2 + d^2)}$ $S = \dfrac{b^2 d^2}{6\sqrt{b^2 + d^2}}$ $r = \dfrac{bd}{\sqrt{6(b^2 + d^2)}}$
RECTANGLE *Axis of moments any line through center of gravity*  	$A = bd$ $c = \dfrac{b \sin a + d \cos a}{2}$ $I = \dfrac{bd\,(b^2 \sin^2 a + d^2 \cos^2 a)}{12}$ $S = \dfrac{bd\,(b^2 \sin^2 a + d^2 \cos^2 a)}{6\,(b \sin a + d \cos a)}$ $r = \sqrt{\dfrac{b^2 \sin^2 a + d^2 \cos^2 a}{12}}$
HOLLOW RECTANGLE *Axis of moments through center*  	$A = bd - b_1 d_1$ $c = \dfrac{d}{2}$ $I = \dfrac{bd^3 - b_1 d_1^3}{12}$ $S = \dfrac{bd^3 - b_1 d_1^3}{6d}$ $r = \sqrt{\dfrac{bd^3 - b_1 d_1^3}{12A}}$ $Z = \dfrac{bd^2}{4} - \dfrac{b_1 d_1^2}{4}$

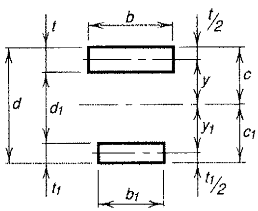
Table 17–27 (continued)
Properties of Geometric Sections

EQUAL RECTANGLES
Axis of moments through center of gravity



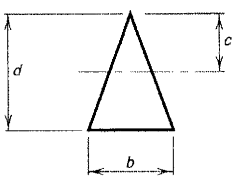
$$A = b(d - d_1)$$
$$c = \frac{d}{2}$$
$$I = \frac{b(d^3 - d_1^3)}{12}$$
$$S = \frac{b(d^3 - d_1^3)}{6d}$$
$$r = \sqrt{\frac{d^3 - d_1^3}{12(d - d_1)}}$$
$$Z = \frac{b}{4}(d^2 - d_1^2)$$

UNEQUAL RECTANGLES
Axis of moments through center of gravity



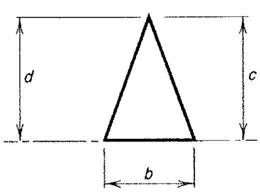
$$A = bt + b_1t_1$$
$$c = \frac{\tfrac{1}{2}bt^2 + b_1t_1(d - \tfrac{1}{2}t_1)}{A}$$
$$I = \frac{bt^3}{12} + bty^2 + \frac{b_1t_1^3}{12} + b_1t_1y_1^2$$
$$S = \frac{I}{c} \qquad S_1 = \frac{I}{c_1}$$
$$r = \sqrt{\frac{I}{A}}$$
$$Z = bty + b_1t_1y_1$$

TRIANGLE
Axis of moments through center of gravity



$$A = \frac{bd}{2}$$
$$c = \frac{2d}{3}$$
$$I = \frac{bd^3}{36}$$
$$S = \frac{bd^2}{24}$$
$$r = \frac{d}{\sqrt{18}} = .235702\,d$$

TRIANGLE
Axis of moments on base



$$A = \frac{bd}{2}$$
$$c = d$$
$$I = \frac{bd^3}{12}$$
$$S = \frac{bd^2}{12}$$
$$r = \frac{d}{\sqrt{6}} = .408248\,d$$

Table 17–27 (continued)
Properties of Geometric Sections

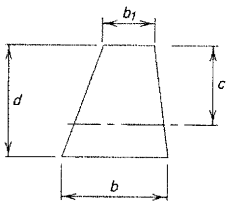
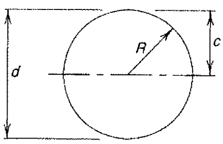
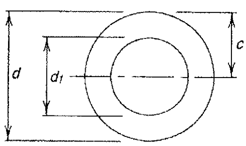
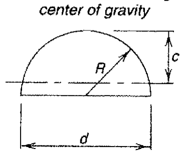
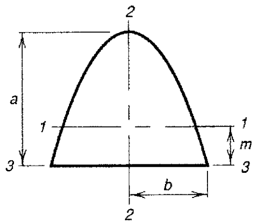
TRAPEZOID *Axis of moments through center of gravity* 	$A = \dfrac{d(b+b_1)}{2}$ $c = \dfrac{d(2b+b_1)}{3(b+b_1)}$ $I = \dfrac{d^3(b^2+4bb_1+b_1^2)}{36(b+b_1)}$ $S = \dfrac{d^2(b^2+4bb_1+b_1^2)}{12(2b+b_1)}$ $r = \dfrac{d}{6(b+b_1)}\sqrt{2(b^2+4bb_1+b_1^2)}$
CIRCLE *Axis of moments through center* 	$A = \dfrac{\pi d^2}{4} = \pi R^2 = .785398\, d^2 = 3.141593\, R^2$ $c = \dfrac{d}{2} = R$ $I = \dfrac{\pi d^4}{64} = \dfrac{\pi R^4}{4} = .049087\, d^4 = .785398\, R^4$ $S = \dfrac{\pi d^3}{32} = \dfrac{\pi R^3}{4} = .098175\, d^3 = .785398\, R^3$ $r = \dfrac{d}{4} = \dfrac{R}{2}$ $Z = \dfrac{d^3}{6}$
HOLLOW CIRCLE *Axis of moments through center* 	$A = \dfrac{\pi(d^2-d_1^2)}{4} = .785398\,(d^2-d_1^2)$ $c = \dfrac{d}{2}$ $I = \dfrac{\pi(d^4-d_1^4)}{64} = .049087\,(d^4-d_1^4)$ $S = \dfrac{\pi(d^4-d_1^4)}{32d} = .098175\,\dfrac{d^4-d_1^4}{d}$ $r = \dfrac{\sqrt{d^2+d_1^2}}{4}$ $Z = \dfrac{d^3}{6} - \dfrac{d_1^3}{6}$
HALF CIRCLE *Axis of moments through center of gravity* 	$A = \dfrac{\pi R^2}{2} = 1.570796\, R^2$ $c = R\left(1 - \dfrac{4}{3\pi}\right) = .575587\, R$ $I = R^4\left(\dfrac{\pi}{8} - \dfrac{8}{9\pi}\right) = .109757\, R^4$ $S = \dfrac{R^3}{24}\,\dfrac{(9\pi^2-64)}{(3\pi-4)} = .190687\, R^3$ $r = R\,\dfrac{\sqrt{9\pi^2-64}}{6\pi} = .264336\, R$

Table 17–27 (continued)
Properties of Geometric Sections

PARABOLA



$$A = \frac{4}{3}ab$$

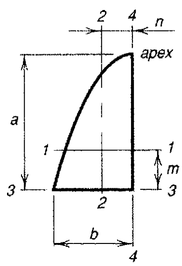
$$m = \frac{2}{5}a$$

$$I_1 = \frac{16}{175}a^3b$$

$$I_2 = \frac{4}{15}ab^3$$

$$I_3 = \frac{32}{105}a^3b$$

HALF PARABOLA



$$A = \frac{2}{3}ab$$

$$m = \frac{2}{5}a$$

$$n = \frac{3}{8}b$$

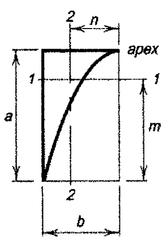
$$I_1 = \frac{8}{175}a^3b$$

$$I_2 = \frac{19}{480}ab^3$$

$$I_3 = \frac{16}{105}a^3b$$

$$I_4 = \frac{2}{15}ab^3$$

COMPLEMENT OF HALF PARABOLA



$$A = \frac{1}{3}ab$$

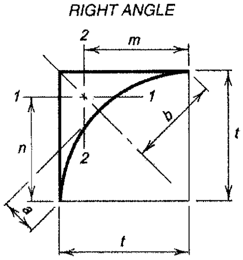
$$m = \frac{7}{10}a$$

$$n = \frac{3}{4}b$$

$$I_1 = \frac{37}{2{,}100}a^3b$$

$$I_2 = \frac{1}{80}ab^3$$

PARABOLIC FILLET IN RIGHT ANGLE



$$a = \frac{t}{2\sqrt{2}}$$

$$b = \frac{t}{\sqrt{2}}$$

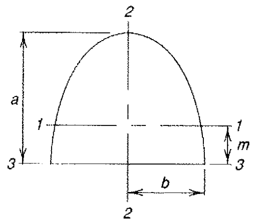
$$A = \frac{1}{6}t^2$$

$$m = n = \frac{4}{5}t$$

$$I_1 = I_2 = \frac{11}{2{,}100}t^4$$

Table 17–27 (continued)
Properties of Geometric Sections

*HALF ELLIPSE



$$A = \frac{1}{2}\pi ab$$

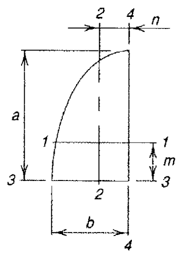
$$m = \frac{4a}{3\pi}$$

$$I_1 = a^3 b\left(\frac{\pi}{8} - \frac{8}{9\pi}\right)$$

$$I_2 = \frac{1}{8}\pi ab^3$$

$$I_3 = \frac{1}{8}\pi a^3 b$$

*QUARTER ELLIPSE



$$A = \frac{1}{4}\pi ab$$

$$m = \frac{4a}{3\pi}$$

$$n = \frac{4b}{3\pi}$$

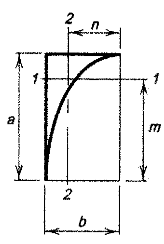
$$I_1 = a^3 b\left(\frac{\pi}{16} - \frac{4}{9\pi}\right)$$

$$I_2 = ab^3\left(\frac{\pi}{16} - \frac{4}{9\pi}\right)$$

$$I_3 = \frac{1}{16}\pi a^3 b$$

$$I_4 = \pi ab^3$$

*ELLIPTIC COMPLEMENT



$$A = ab\left(1 - \frac{\pi}{4}\right)$$

$$m = \frac{a}{6\left(1 - \frac{\pi}{4}\right)}$$

$$n = \frac{b}{6\left(1 - \frac{\pi}{4}\right)}$$

$$I_1 = a^3 b\left(\frac{1}{3} - \frac{\pi}{16} - \frac{1}{36\left(1 - \frac{\pi}{4}\right)}\right)$$

$$I_2 = ab^3\left(\frac{1}{3} - \frac{\pi}{16} - \frac{1}{36\left(1 - \frac{\pi}{4}\right)}\right)$$

*To obtain properties of half circle, quarter circle, and circular complement, substitute $a = b = R$.

Table 17–27 (continued)
Properties of Geometric Sections

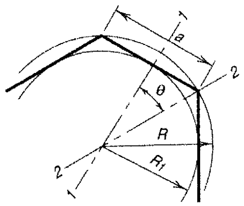
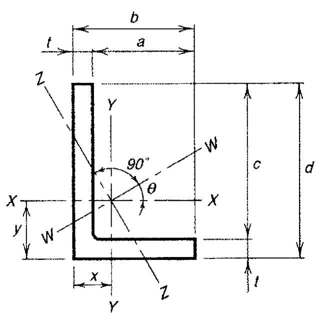
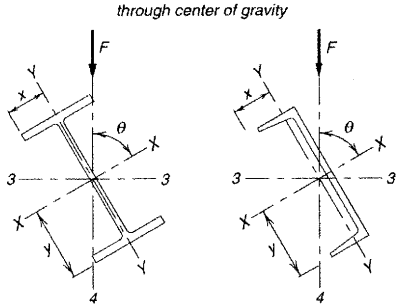
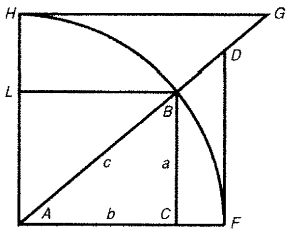
REGULAR POLYGON *Axis of moments through center* 	n = Number of sides $\theta = \frac{180°}{n}$ $a = 2\sqrt{R^2 - R_1^2}$ $R = \frac{a}{2 \sin \theta}$ $R_1 = \frac{a}{2 \tan \theta}$ $A = \frac{1}{4} na^2 \cot \theta = \frac{1}{2} nR^2 \sin 2\theta = nR_1^2 \tan \theta$ $I_1 = I_2 = \frac{A(6R^2 - a^2)}{24} = \frac{A(12R_1^2 + a^2)}{48}$ $r_1 = r_2 = \sqrt{\frac{6R^2 - a^2}{24}} = \sqrt{\frac{12R_1^2 + a^2}{48}}$
ANGLE *Axis of moments through center of gravity* 	$\tan 2\theta = \frac{2K}{I_Y - I_X}$ $A = t(b + c)\ x = \frac{b^2 + ct}{2(b + c)}\ y = \frac{d^2 + at}{2(b + c)}$ K = Product of Inertia about X X and Y Y $= \pm \frac{abcdt}{4(b + c)}$ $I_X = \frac{1}{3}(t(d - y)^3 + by^3 - a(y - t)^3)$ $I_Y = \frac{1}{3}(t(b - x)^3 + dx^3 - c(x - t)^3)$ $I_Z = I_X \sin^2 \theta + I_Y \cos^2 \theta + K \sin 2\theta$ $I_W = I_X \cos^2 \theta + I_Y \sin^2 \theta - K \sin 2\theta$ K is negative when heel of angle, with respect to center of gravity, is in 1st or 3rd quadrant, positive when in 2nd or 4th quadrant.
BEAMS AND CHANNELS *Transverse force oblique through center of gravity* 	$I_3 = I_X \sin^2 \theta + I_Y \cos^2 \theta$ $I_4 = I_X \cos^2 \theta + I_Y \sin^2 \theta$ $f_b = M\left(\frac{y}{I_X} \sin \theta + \frac{x}{I_Y} \cos \theta\right)$ where M is bending moment due to force F.

Table 17–28
Trigonometric Formulas

TRIGNOMETRIC FUNCTIONS



Radius $AF = 1$

$$= \sin^2 A + \cos^2 A = \sin A \operatorname{cosec} A$$

$$= \cos A \sec A = \tan A \cot A$$

$$\sin A = \frac{\cos A}{\cot A} = \frac{1}{\operatorname{cosec} A} = \cos A \tan A = \sqrt{1-\cos^2 A} = BC$$

$$\cos A = \frac{\sin A}{\tan A} = \frac{1}{\sec A} = \sin A \cot A = \sqrt{1-\sin^2 A} = AC$$

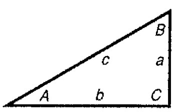
$$\tan A = \frac{\sin A}{\cos A} = \frac{1}{\cot A} = \sin A \sec A = FD$$

$$\cot A = \frac{\cos A}{\sin A} = \frac{1}{\tan A} = \cos A \operatorname{cosec} A = HG$$

$$\sec A = \frac{\tan A}{\sin A} = \frac{1}{\cos A} = AD$$

$$\operatorname{cosec} A = \frac{\cot A}{\cos A} = \frac{1}{\sin A} = AG$$

RIGHT ANGLED TRIANGLES



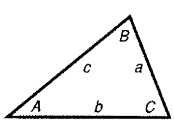
$$a^2 = c^2 - b^2$$

$$b^2 = c^2 - a^2$$

$$c^2 = a^2 + b^2$$

Known	Required A	B	a	b	c	Area
a, b	$\tan A = \frac{a}{b}$	$\tan B = \frac{b}{a}$			$\sqrt{a^2+b^2}$	$\frac{ab}{2}$
a, c	$\sin A = \frac{a}{c}$	$\cos B = \frac{a}{c}$		$\sqrt{c^2-a^2}$		$\frac{a\sqrt{c^2-a^2}}{2}$
A, a		$90° - A$		$a \cot A$	$\frac{a}{\sin A}$	$\frac{a^2 \cot A}{2}$
A, b		$90° - A$	$b \tan A$		$\frac{b}{\cos A}$	$\frac{b^2 \tan A}{2}$
A, c		$90° - A$	$c \sin A$	$c \cos A$		$\frac{c^2 \sin 2A}{4}$

OBLIQUE ANGLED TRIANGLES



$$s = \frac{a+b+c}{2}$$

$$K = \sqrt{\frac{(s-a)\,(s-b)\,(s-c)}{s}}$$

$$a^2 = b^2 + c^2 - 2bc \cos A$$

$$b^2 = a^2 + c^2 - 2ac \cos B$$

$$c^2 = a^2 + b^2 - 2ab \cos C$$

Known	Required A	B	C	b	c	Area
a, b, c	$\tan \frac{1}{2} A = \frac{K}{s-a}$	$\tan \frac{1}{2} B = \frac{K}{s-b}$	$\tan \frac{1}{2} C = \frac{K}{s-c}$			$\sqrt{s(s-a)(s-b)(s-c)}$
a, A, B			$180° - (A+B)$	$\frac{a \sin B}{\sin A}$	$\frac{a \sin C}{\sin A}$	
a, b, A		$\sin B = \frac{b \sin A}{a}$			$\frac{b \sin C}{\sin B}$	
a, b, C	$\tan A = \frac{a \sin C}{b - a \cos C}$				$\sqrt{a^2+b^2-2ab \cos C}$	$\frac{ab \sin C}{2}$

GENERAL NOMENCLATURE

The following definitions apply, as these variables are used in this Manual. Additional nomenclature used in both the Manual and the Specification can be found in the AISC *Specification for Structural Steel Buildings*, in Part 16 of this Manual.

A	Area of directly connected elements, in.^{2}
A	Horizontal distance from end panel point to mid-span of a truss, ft
A	Minimum side dimension for square or rectangular beveled washer, in.
A_b	Required transverse force from an adjacent bay, kips.
A_{cp}	Projected surface area of concrete cone surrounding headed anchor rods, in.^{2}
A_f	Area of flange, in.^{2}
A_{fe}	Effective tension flange area, in.^{2}
A_{gt}	Gross area subject to tension, in.^{2}
B	Available tensile strength per bolt subjected to prying action, kips.
B	Horizontal distance from mid-span of a truss to a given panel point, ft
B	Base plate width, in.
BF	A factor that can be used to calculate the flexural strength for unbraced length L_b between L_p and L_r
C	Required mid-span camber, in.
C	Width across points of square or hex bolt head or nut, or maximum diameter of countersunk bolt head, in.
C	Coefficient for eccentrically loaded bolt and weld groups
C_{Tot}	Sum of compressive forces in a composite beam, kips
C_c	Beam reaction coefficient
C_{conc}	Effective concrete flange force for a composite beam, kips
C_{stl}	Compressive force in steel in a composite beam, kips
C_1	Loading constant used in deflection calculations
C_1	Clearance for tightening, in.
C_1	Electrode coefficient for relative strength of electrodes where, for E70 electrodes, $C_1 = 1.00$
C_2	Clearance for entering, in.
C_3	Clearance for fillet based on one standard hardened washer, in.
C'	Coefficient for eccentrically loaded bolt groups subjected to moment only
CG	Center of gravity
D	Offset from the base line at a panel point of a truss, in.
D	Number of sixteenths-of-an-inch in the weld size
E	Earthquake load
E	Minimum edge distance for clipped washer, in.
E	Minimum effective throat thickness for partial-joint-penetration groove weld, in.
ENA	Elastic neutral axis
F	Width across flats of bolt head, in.
F	Clearance for tightening staggered bolts, in.
F_p	Nominal bearing stress on fastener, ksi

F_{yb}	F_y of a beam, ksi
F_{yc}	F_y of a column, ksi
F_{yc}	F_y of a cap plate, ksi
F_{yf}	Specified minimum yield stress of the flange, ksi
G	Ratio of the total column stiffness framing into a joint to that of the stiffening members framing into the same joint
H	Horizontal force, kips
H	Height of bolt head or nut, in.
H	Theoretical thread height, in.
H_b	Required shear force on a beam to gusset connection, kips
H_c	Required axial force on a column go gusset connection, kips
H_1	Height of bolt head, in.
H_2	Maximum bolt shank extension based on one standard hardened washer, in.
I_{LB}	Lower bound moment of inertia for composite section, in.^{4}
I_c	Moment of inertia of column section about axis perpendicular to plane of buckling, in.^{4}
I_g	Moment of inertia of girder about axis perpendicular to plane of buckling, in.^{4}
I_p	Moment of inertia of primary member, in.^{4}
I_p	Polar moment of inertia of bolt and weld groups ($I_p = I_x + I_y$), in.^{4} per in.^{2}
I_{st}	Moment of inertia of a transverse stiffener, in.^{4}
I_x	Moment of inertia of bolt and weld groups about x-axis, in.^{4} per in.^{2}
I_y	Moment of inertia of bolt and weld groups about y-axis, in.^{4} per in.^{2}
I_{yc}	Moment of inertia about y-axis referred to compression flange, or if reverse curvature bending referred to smaller flange, in.^{4}
IC	Instantaneous center of rotation
ID	Nominal inside diameter of flat circular washer, in.
K	Minimum root diameter of threaded fastener, in.
K_{dep}	Fillet depth, $(k - t_f)$, in.
L	Length of connection in the direction of loading, in.
L	Live load due to occupancy and moveable equipment
L_c	Unsupported length of a column section, ft
L_e	Edge distance, in.
L_{eh}	Horizontal edge distance, in.
L_{ev}	Vertical edge distance, in.
L_g	Unsupported length of a girder or other restraining member, ft
L_h	Hook length for hooked anchor rods, in.
$L_p{}'$	Limiting laterally unbraced length for the maximum design flexural strength for noncompact shapes, uniform moment case ($C_b = 1.0$), in. or ft, as indicated
L_r	Roof live load
M	Beam bending moment, kip-in. or kip-ft, as indicated
M_{LL}	Beam moment due to live load, kip-in. or kip-ft, as indicated
M_{cr}	Elastic buckling moment, kip-in. or kip-ft, as indicated
$M_p{}'$	Maximum available flexural strength for noncompact shapes, when $L_b \le L_p{}'$, kip-in. or kip-ft, as indicated
M_{pa}	Plastic bending moment modified by axial load ratio, kip-in.
M_r	Limiting buckling moment, M_{cr}, when $\lambda = \lambda_r$ and $C_b = 1.0$, kip-in. or kip-ft, as indicated

M_u Required flexural strength using LRFD load combinations, kip-in. or kip-ft, as indicated

N Length of base plate, in.

N_b Number of bolts in a joint

N_r Number of shear stud connectors in one rib at a beam intersection.

N_r Required length of bearing, in.

OD Nominal outside diameter of flat circular washer, in.

P Concentrated load, kips

P Required axial force, kips

P Bolt stagger, in.

P_a Required concentrated beam load using ASD load combinations, kips

P_a Required axial strength (tension or compression) using ASD load combinations, kips

P_{af} Required beam flange force, tensile or compressive, using ASD load combinations, kips

P_e Elastic (Euler) buckling load, kips

P_{fb} Resistance to flange local bending per AISC Specification Equation J10-1 (used to check need for column web stiffeners), kips

P_u Required concentrated beam load using LRFD load combinations, kips

P_{uf} Factored beam flange force, tensile or compressive, using LRFD load combinations, kips

P_{wb} Resistance to web compression buckling per AISC Specification Equation J10-8 (used to check need for column web stiffening), kips

P_{wi} A factor consisting of terms from the second portion of AISC Specification Equation J10-2 (used in a column web stiffener check for web local yielding), kips/in.

P_{wo} A factor consisting of the first portion of AISC Specification Equation J10-2 (used in a column web stiffener check for web local yielding), kips

PNA Plastic neutral axis

R Nominal reaction, kips

R Required end reaction, kips

$R_{a\,st}$ Required strength for transverse stiffener (force delivered to stiffener) using ASD load combinations, kips

$R_{u\,st}$ Required strength for transverse stiffener (force delivered to stiffener) using LRFD load combinations, kips

R_v Web shear strength, kips

R_w Effective nominal strength of a concentrically loaded weld group, kips

R_1 Beam bearing constant, see Part 9.

R_2 Beam bearing constant, see Part 9.

R_3 Beam bearing constant, see Part 9.

R_4 Beam bearing constant, see Part 9.

R_5 Beam bearing constant, see Part 9.

R_6 Beam bearing constant, see Part 9.

S Spacing, in. or ft, as indicated

S Groove depth for partial-joint-penetration groove welds, in.

S_{net} Net elastic section modulus, $in.^3$

T Distance between web toes of fillets at top and at bottom of web, in. $= d - 2k$

T Tension force due to service loads, kips

T	Thickness of flat circular washer or mean thickness of square or rectangular beveled washer, in.
T_{avail}	Available tensile strength, kips
T_{stl}	Tensile force in steel in a composite beam, kips
T_{Tot}	Sum of tensile forces in a composite beam, kips
V	Shear force, kips
V_a	Required shear strength using ASD load combinations, kips
V_b	Shear force component, kips
V_b	Required shear force on a beam to gusset connection, kips
V_u	Required shear strength using LRFD load combinations, kips
W	Wind load
W	Uniformly distributed load, kips
W	Weight, lbs or kips, as indicated
W	Width across flats of nut, in.
W_a	Total factored uniformly distributed load using ASD load combinations, kips
W_c	Uniform load constant for beams, kip-ft
W_u	Total factored uniformly distributed load using LRFD load combinations, kips
Y_{ENA}	Distance from bottom of steel beam to elastic neutral axis, in.
Y_{con}	Distance from top of steel beam to top of concrete, in.
Y_1	Distance from top of steel beam to the plastic neutral axis, in.
Y_2	Distance from top of steel beam to the concrete flange force in a composite beam, in.
Z_e	Effective plastic section modulus, in.^{3}
Z_{net}	Net plastic section modulus, in.^{3}
a	Effective concrete flange thickness of a composite beam, in.
a	Coefficient for eccentrically loaded weld group
a	Distance from bolt centerline to edge of fitting subjected to prying action, but not greater than 1.25b, in.
a	Distance from an HSS centroid to the end of an attached member, in.
a	Distance from the weld line to the first row of bolts in a single plate connection, in.
a'	Weld length, in.
b	Effective concrete flange width in a composite beam, in.
b	Minimum shelf dimension for deposition of fillet weld, in.
b	Distance from bolt centerline to face of fitting subjected to prying action, in.
b_{eff}	Effective width, in.
b_x	Coefficient for strong axis bending related to combined axial and bending strength calculations (see Part 6)
b_y	Coefficient for weak axis bending related to combined axial and bending strength calculations (see Part 6)
c	Distance from the neutral axis to the extreme fiber of the cross section, in.
c	Radial distance from the center of gravity to the portion of the weld group most remote from the center of gravity, in.
c	Cope length, in.
d_c	Cope depth, in.
d_{ct}	Top-flange cope depth, in.
d_{cb}	Bottom-flange cope depth, in.
d_h	Hole diameter, in.

d_m	Moment arm between resultant tensile and compressive forces due to a moment or eccentric force, in.
d_w	Diameter of a part in contact with the inner surface of an HSS, in.
d_z	Overall panel-zone depth, in.
e	Eccentricity, in.
e	Base of natural logarithms = 2.71828...
e_o	Horizontal distance from the outer edge of a channel web to its shear center, in.
f	Computed compressive stress in the stiffened element, ksi
f	Plate buckling model adjustment factor for beams coped at top flange only
f_b	Maximum bending stress, ksi
f_d	Adjustment factor for beams coped at both flanges
f_{un}	Required normal stress, ksi
f_{uv}	Required shear stress, ksi
g	Transverse center-to-center spacing (gage) between fastener gage lines, in.
g	Acceleration due to gravity = 32.2 ft/sec^2 = 386 in./sec^2
h_o	Distance between flange centroids, in.
h_r	Nominal rib height, in.
h_o	Remaining web depth of coped beam, in.
k	Plate buckling coefficient for beams coped at top flange only
k_1	Distance from web center line to flange toe of fillet, in.
kip	1,000 pounds
ksi	kips/in.^{2}
l	Span length, in.
l	Length of weld, in.
l	Characteristic length of weld group, in.
l_i	Distance of the ith bolt from the center of gravity, in.
l_o	Distance from center of gravity (CG) to instantaneous center of rotation (IC) of bolt or weld group, in.
m	Cantilever dimension for base plate, in.
n	Number of shear connectors between point of maximum positive moment and the point of zero moment to each side
n	Number of bolts in a vertical row
n	Cantilever dimension for base plate, in.
n'	Number of bolts above the neutral axis (in tension)
p	Length of supporting flange parallel to stem or leg of hanger tributary to each bolt in determining prying action, in.
p	Coefficient for axial compression related to combined axial and bending strength calculations (see Part 6)
q	Horizontal shear, kips/in.
q	Additional tension per bolt resulting from prying action produced by deformation of the connected parts, kips/bolt
r_a	Required shear strength per bolt using ASD load combinations, kips/bolt
r_{at}	Required tensile strength per bolt or per inch of weld using ASD load combinations (force per bolt or per inch of weld due to a tensile force), kips/bolt
r_{av}	Required shear strength per bolt or per inch of weld using ASD load combinations (force per bolt or per inch of weld due to a shear force), kips/bolt
r_m	Radius of gyration of steel shape, pipe, or tubing in composite columns.

r_m	Required shear force on the bolt most remote from the center of gravity, due to moment, kips
r_m	Shear per inch of weld due to moment, kips/in.
r_n	Nominal strength per bolt, kips
r_p	Required shear strength per bolt due to a concentric force, kips/bolt
r_u	Required shear strength per bolt using LRFD load combinations, kips/bolt
r_{ut}	Required tensile strength per bolt or per inch of weld using LRFD load combinations (force per bolt or per inch of weld due to a tensile force), kips/bolt
r_{uv}	Required shear strength per bolt or per inch of weld using LRFD load combinations (force per bolt or per inch of weld due to a shear force), kips/bolt
r_x, r_y	Radius of gyration about x and y axes respectively, in.
r_{yc}	Radius of gyration about y axis referred to compression flange, or if reverse curvature bending, referred to smaller flange, in.
$\bar{r}_o$	Polar radius of gyration about the shear center, in.
t	Change in temperature, degrees Fahrenheit or Celsius, as indicated
t_b	Thickness of beam flange or connection plate delivering concentrated force, in.
t_c	Flange or angle thickness required to develop design tensile strength of bolts with no prying action, in.
t_c	Thickness of cap plate, in.
t_{des}	Design thickness of an HSS wall, in.
t_f	Lesser connection element thickness, in.
t_{nom}	Nominal thickness of an HSS wall, in.
t_r	Coefficient for tension rupture related to combined axial and bending strength calculations (see Part 6)
t_s	Tee stem thickness, in.
t_{wb}	Beam web thickness, in.
t_{wc}	Column web thickness, in.
t_y	Coefficient for tension yielding related to combined axial and bending strength calculations (see Part 6)
w	Uniformly distributed load per unit of length, kips/in.
w	Plate width; distance between welds, in.
x	Horizontal distance, in.
x	Horizontal distance from the support to the location of applied bearing force, in.
$\bar{x}$	Horizontal distance from the outer edge of a channel web to its centroid, in.
x_p	Horizontal distance from the designated edge of member to its plastic neutral axis, in.
x_o	Horizontal distance, in.
y	Moment arm between centroid of tensile forces and compressive forces, in.
y_p	Vertical distance from the designated edge of member to its plastic neutral axis, in.
y_1, y_2	Vertical distance from designated edge of member to center of gravity, in.
z	Coefficient for buckling of triangular-shaped bracket plate
Δ	Deflection, in.
α	Fraction of member force transferred across a particular net section
α	Ratio of moment at bolt line to moment at stem line for determining prying action in hanger connections
α	Ideal distance from face of column flange or web to centroid of gusset-to-beam connection for bracing connections and uniform force method, in.

$\bar{\alpha}$ Actual distance from face of column flange or web to centroid of gusset-to-beam connection for bracing connections and uniform force method, in.

β Ideal distance from face of beam flange to centroid of gusset-to-column connection for bracing connections and uniform force method, in.

$\bar{\beta}$ Actual distance from face of beam flange to centroid of gusset-to-column connection for bracing connections and uniform force method, in.

δ Deflection, in.

δ Ratio of net area at bolt line to gross area at face of stem or angle leg used to determine prying action for hanger connections

ε Coefficient of linear expansion, with units as indicated

τ_a Stiffness reduction factor, for use with the alignment charts (AISC Specification Figures C-C2.3 and C-C2.4) in the determination of effective length factors, K, for columns

ν Poisson's ratio = 0.3 for steel

ϕR_n Design strength from AISC Specification; must equal or exceed required strength using LRFD load combinations, R_u

ϕr_n Design strength per bolt or per inch of weld from AISC Specification; must equal or exceed required strength per bolt or per inch of weld using LRFD load combinations, r_u

R_n/Ω Allowable strength from AISC Specification; must equal or exceed required strength using ASD load combinations, R_a

r_n/Ω Allowable strength per bolt or per inch of weld from AISC Specification; must equal or exceed required strength per bolt or per inch of weld using ASD load combinations, r_a

INDEX

The following list of terms provides reference to items found in the AISC *Steel Construction Manual*, as well as selected supporting references. The locations of supporting references have been abbreviated as follows:

> "DG#" is used for items found in AISC's Design Guide series.
> "SDM" is used for items found in the AISC *Seismic Design Manual.*
> "DSC" is used for items found in AISC's *Detailing for Steel Construction.*
> "AISC Design Examples" indicates that information can be found on the CD companion to this AISC *Steel Construction Manual.*