SUMMARY OF MAJOR CHANGES IN PUBLIC REVIEW ONE OF AISC 360-22

Substantial changes to ANSI/AISC 360 in the 2022 edition that appear in Public Review One Draft dated August 3, 2020:

- New shear lag factors are provided for slotted round and rectangular
- HSS members connected to a gusset plate and for rectangular HSS members connected two side gusset plates.
- New provisions are provided compression members with lateral bracing offset from the shear center (also known as constrained axis torsional buckling).
- Eurocode stress-strain-temperature equations have been incorporated in Appendix 4 (fire) so users have clearer guidance on that material properties they can use for steel and concrete at elevated temperatures.
- Appendix 4, Section 4.3, "Design by Qualification Testing," now includes prescriptive steel fire protection design equations and related information based on standard ASTM E119 fire tests, which have also been contained in ASCE-29 and the IBC.
- Sections A4, Structural Design Documents and Specifications, has been expanded to list information from the Code of Standard Practice that needs to be provided in the structural design documents.
- A new Section A5, Approvals, has been added to address the review and approval of approval documents.
- Chapter I, "Design of Composite Members," has been expanded to include the coupled concrete filled composite plate shear wall system.
- New provisions added to Chapter I, "Design of Composite Members," has made this chapter the single source standard for the design of composite members and systems.
- New provisions have been added for both filled and encased members.
- A new Appendix has been added to allow for the design of filled composite members with higher strength materials ($f'c \leq$ 15,000 psi and $F_y \leq$ 100 ksi) ASTM F3148 (144 ksi) bolts have been added to the Specification.

Specification for Structural Steel Buildings

Public Review Draft dated August 3, 2020

Supersedes the *Specification for Structural Steel Buildings* dated July 7, 2016 and all previous versions of this specification

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Symbols

Definitions for the symbols used in this standard are provided here and reflect the definitions provided in the body of this standard. Some symbols may be used multiple times throughout the document. The section or table number shown in the right-hand column of the list identifies the first time the symbol is used in this document. Symbols without text definitions are omitted.

Symbol	Definition	Section
A_{BM}	Area of the base metal, in.^{2} (mm^2)	J2.4
A_b	Nominal unthreaded body area of bolt or threaded part, in.^{2} (mm^2)	J3.6
A_b	Nominal body area of undriven rivet, in.^{2} (mm^2)	App. 5.3.2a
A_c	Area of concrete, in.^{2} (mm^2)	I2.1b
A_c	Area of concrete slab within effective width, in.^{2} (mm^2)	I3.2d
A_c	Area of concrete in filled composite member, in.^{2} (mm^2)	I4.2
A_e	Effective area, in.^{2} (mm^2)	E7.2
A_e	Effective net area, in.^{2} (mm^2)	D2
A_e	Effective net area as defined in Section D3, in.^{2}(mm^2)	J4.1
A_e	Summation of the effective areas of the cross section based on the reduced effective widths, b_e, d_e or h_e, or the area as given by Equations E7-6 or E7-7, in.^{2} (mm^2)	E7
A_{ev}	Effective area subjected to shear, in.^{2} (mm^2)	J4.3
A_{fc}	Area of compression flange, in.^{2} (mm^2)	G2.2
A_{fg}	Gross area of tension flange, calculated in accordance with Section B4.3a, in.^{2} (mm^2)	F13.1
A_{fn}	Net area of tension flange, calculated in accordance with Section B4.3b, in.^{2} (mm^2)	F13.1
A_{ft}	Area of tension flange, in.^{2} (mm^2)	G2.2
A_g	Gross area of angle, in.^{2} (mm^2)	F10.2
A_g	Gross area of member, in.^{2} (mm^2)	B4.3a
A_g	Gross area of eyebar body, in.^{2} (mm^2)	D6.1
A_g	Gross area of composite member, in.^{2} (mm^2)	I2.1a
A_{gv}	Gross area subject to shear, in.^{2} (mm^2)	J4.2
A_n	Net area of member, in.^{2} (mm^2)	B4.3b
A_{nt}	Net area subject to tension, in.^{2} (mm^2)	J4.3
A_{nv}	Net area subject to shear, in.^{2} (mm^2)	J4.2
A_{pb}	Projected area in bearing, in.^{2} (mm^2)	J7
A_s	Area of steel section, in.^{2} (mm^2)I1.5	
A_{sa}	Cross-sectional area of steel headed stud anchor, in.^{2} (mm^2)	I8.2a
A_{sf}	Area on the shear failure path, in.^{2} (mm^2)	D5.1
A_{sr}	Area of continuous reinforcing bars, in.^{2} (mm^2)	I2.1a
A_{sr}	Area of developed longitudinal reinforcing steel within the effective width of the concrete slab, in.^{2} (mm^2)	I3.2d.2
A_{sw}	Area of steel plates in the direction of in-plane shear, in.^{2} (mm^2)	I1.5
A_t	Net area in tension, in.^{2} (mm^2)	App. 3.4
A_T	Nominal forces and deformations due to the design-basis fire defined in Section 4.2.1	App. 4.1.4
A_v	Shear area of the steel portion of a composite member. The shear area for a circular section is equal to $2A_s/\pi$, and for a rectangular section is equal to the sum of the area of webs in the direction of in-plane shear, in.^{2} (mm^2)	I4.2

A_w	Area of web, the overall depth times the web thickness, dt_w, in.^{2} (mm^2)	G2.1
A_w	Area of web or webs, taken as the sum of the overall depth times the web thickness, dt_w, in.^{2} (mm^2)	G4
A_{we}	Effective area of the weld, in.^{2} (mm^2)	J2.4
A_1	Loaded area of concrete, in.^{2} (mm^2)	I6.3a
A_1	Area of steel concentrically bearing on a concrete support, in.^{2} (mm^2)	J8
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)	J8
B	Overall width of rectangular HSS main member, measured 90° to the plane of the connection, in. (mm)	Table D3.1
B_b	Overall width of rectangular HSS branch member or plate, measured 90° to the plane of the connection, in. (mm)	K1.1
B_e	Effective width of rectangular HSS branch member or plate for local yielding of the transverse element, in. (mm)	K1.1
B_{ep}	Effective width of rectangular HSS branch member or plate for punching shear, in. (mm)	K1.1
B_1	Multiplier to account for P-δ effects	App. 8.2
B_2	Multiplier to account for P-Δ effects	App. 8.2
C	HSS torsional constant	H3.1
C_b	Lateral-torsional buckling modification factor for nonuniform moment diagrams when both ends of the segment are braced	F1
C_f	Constant from Table A-3.1 for the fatigue category	App. 3.3
C_m	Equivalent uniform moment factor assuming no relative translation of member ends	App. 8.2.1
C_{v1}	Web shear strength coefficient	G2.1
C_{v2}	Web shear buckling coefficient	G2.2
C_w	Warping constant, in.^{6} (mm^6)	E4
C_1	Coefficient for calculation of effective rigidity of encased composite compression member	I2.1b
C_2	Edge distance increment, in. (mm)	Table J3.5
C_3	Coefficient for calculation of effective rigidity of filled composite compression member	I2.2b
D	Diameter of round HSS, in. (mm)	B4.1b
D	Outside diameter of round HSS, in. (mm)	Table D3.1
D	Outside diameter of round HSS main member, in. (mm)	K1.1
D	Nominal dead load, kips (N)	B3.9
D	Nominal dead load rating	App. 5.4.1
D_b	Outside diameter of round HSS branch member, in. (mm)	K1.1
D_u	A multiplier that reflects the ratio of the mean installed bolt pretension to the specified minimum bolt pretension	J3.8
E	Modulus of elasticity of steel = 29,000 ksi (200 000 MPa)	Table B4.1b
E_c	Modulus of elasticity of concrete = $w_c^{1.5}\sqrt{f_c'}$, ksi ($0.043w_c^{1.5}\sqrt{f_c'}$, MPa)	I2.1b
E_s	Modulus of elasticity of steel = 29,000 ksi (200 000 MPa)	I2.1b
EI_{eff}	Effective stiffness of composite section, kip-in.^{2} (N-mm^2)	I2.1b
F_c	Available stress in main member, ksi (MPa)	K1.1
F_{ca}	Available axial stress at the point of consideration, determined in accordance with Chapter E for compression or Section D2 for tension, ksi (MPa)	H2
F_{cbw}, F_{cbz}	Available flexural stress at the point of consideration, determined in accordance with Chapter F, ksi (MPa)	H2
F_{cr}	Buckling stress for the section as determined by analysis, ksi (MPa)	H3.3

F_{cr} Critical stress, ksi (MPa) E3

F_{cr} Lateral-torsional buckling stress for the section as determined by analysis, ksi (MPa) F12.2

F_{cr} Local buckling stress for the section as determined by analysis, ksi (MPa) F12.3

F_{cr} Critical buckling stress for steel element of filled composite members, ksi (MPa) I6.2b

F_e Elastic buckling stress, ksi (MPa) E3

F_{el} Elastic local buckling stress determined according to Equation E7-5 or an elastic local buckling analysis, ksi (MPa) E7.1

F_{EXX} Filler metal classification strength, ksi (MPa) J2.4

F_{in} Nominal bond stress, ksi (MPa) I6.3c

F_L Nominal compression flange stress above which the inelastic buckling limit states apply, ksi (MPa) F4.2

F_n Nominal tensile stress, F_{nt}, or shear stress, F_{nv}, from Table J3.2, ksi (MPa) J3.6

F_{nBM} Nominal stress of the base metal, ksi (MPa) J2.4

F_{nt} Nominal tensile stress from Table J3.2, ksi (MPa) J3.6

F_{nt} Nominal tensile strength of the driven rivet from Table A-5.3.1, ksi (MPa) .... App. 5.3.2a

F'_{nt} Nominal tensile stress modified to include the effects of shear stress, ksi (MPa) J3.7

F_{nv} Nominal shear stress from Table J3.2, ksi (MPa) J3.6

F_{nv} Nominal shear strength of the driven rivet from Table A-5.3.1, ksi (MPa) .... App. 5.3.2a

F_{nw} Nominal stress of the weld metal, ksi (MPa) J2.4

F_{nw} Nominal stress of the weld metal in accordance with Chapter J, ksi (MPa) K5

F_{SR} Allowable stress range, ksi (MPa) .... App. 3.3

F_{TH} Threshold allowable stress range, maximum stress range for indefinite design life from Table A-3.1, ksi (MPa) .... App. 3.3

F_u Specified minimum tensile strength, ksi (MPa) D2

F_u Specified minimum tensile strength of a steel headed stud anchor, ksi (MPa) I8.2a

F_u Specified minimum tensile strength of the connected material, ksi (MPa) J3.10

F_u Specified minimum tensile strength of HSS member material, ksi (MPa) K1.1

F_y Specified minimum yield stress, ksi (MPa). As used in this Specification, “yield stress” denotes either the specified minimum yield point (for those steels that have a yield point) or specified yield strength (for those steels that do not have a yield point) .... Table B4.1b

F_y Specified minimum yield stress of the type of steel being used, ksi (MPa) E3

F_y Specified minimum yield stress of the column web, ksi (MPa) J10.6

F_y Specified minimum yield stress of HSS main member material, ksi (MPa) K1.1

F_{yb} Specified minimum yield stress of HSS branch member or plate material, ksi (MPa) K1.1

F_{yf} Specified minimum yield stress of the flange, ksi (MPa) J10.1

F_{ysr} Specified minimum yield stress of reinforcing steel, ksi (MPa) I2.1b

F_{yst} Specified minimum yield stress of the stiffener material, ksi (MPa) G2.4

F_{yw} Specified minimum yield stress of the web material, ksi (MPa) G2.3

G Shear modulus of elasticity of steel = 11,200 ksi (77 200 MPa) E4

G_c Shear modulus of concrete, ksi (MPa) I1.5

H	Flexural constant	E4
H	Maximum transverse dimension of rectangular steel member, in. (mm)	I6.3c
H	Total story shear, in the direction of translation being considered, produced by the lateral forces used to compute Δ_H, kips (N)	App. 8.2.2
H	Overall height of rectangular HSS member, measured in the plane of the connection, in. (mm)	K1.1
H_b	Overall height of rectangular HSS branch member, measured in the plane of the connection, in. (mm)	K1.1
I	Moment of inertia in the plane of bending, in.^{4} (mm^4)	App. 8.2.1
I_c	Moment of inertia of the concrete section about the elastic neutral axis of the composite section, in.^{4} (mm^4)	I1.5
I_s	Moment of inertia of steel shape about the elastic neutral axis of the composite section, in.^{4} (mm^4)	I1.5
I_{sr}	Moment of inertia of reinforcing bars about the elastic neutral axis of the composite section, in.^{4} (mm^4)	I2.1b
I_{st}	Moment of inertia of transverse stiffeners about an axis in the web center for stiffener pairs, or about the face in contact with the web plate for single stiffeners, in.^{4} (mm^4)	G2.4
I_{st1}	Minimum moment of inertia of transverse stiffeners required for development of the full shear post buckling resistance of the stiffened web panels, in.^{4} (mm^4)	G2.4
I_{st2}	Minimum moment of inertia of transverse stiffeners required for development of web shear buckling resistance, in.^{4} (mm^4)	G2.4
I_x, I_y	Moment of inertia about the principal axes, in.^{4} (mm^4)	E4
I_y	Moment of inertia about the *y*-axis, in.^{4}(mm^4)	F2.2
I_{yeff}	Effective out-of-plane moment of inertia, in.^{4} (mm^4)	App. 6.3.2a
I_{yc}	Moment of inertia of the compression flange about the *y*-axis, in.^{4} (mm^4)	F4.2
I_{yt}	Moment of inertia of the tension flange about the *y*-axis, in.^{4} (mm^4)	App. 6.3.2a
J	Torsional constant, in.^{4} (mm^4)	E4
K	Effective length factor	E2
K_x	Effective length factor for flexural buckling about *x*-axis	E4
K_y	Effective length factor for flexural buckling about *y*-axis	E4
K_z	Effective length factor for torsional buckling about the longitudinal axis	E4
L	Length of member, in. (mm)	H3.1
L	Laterally unbraced length of member, in. (mm)	E2
L	Length of span, in. (mm)	App. 6.3.2a
L	Length of member between work points at truss chord centerlines, in. (mm)	E5
L	Nominal live load	B3.9
L	Nominal live load rating	App. 5.4.1
L	Nominal occupancy live load, kips (N)	App. 4.1.4
L	Height of story, in. (mm)	App. 7.3.2
L_b	Length between points that are either braced against lateral displacement of compression flange or braced against twist of the cross section, in. (mm)	F2.2
L_b	Laterally unbraced length of member, in. (mm)	F10.2
L_b	Length between points that are either braced against lateral displacement of the compression region, or between points braced to prevent twist of the cross section, in. (mm)	F11.2
L_b	Largest laterally unbraced length along either flange at the point of load, in. (mm)	J10.4
L_{br}	Unbraced length within the panel under consideration, in. (mm)	

.... App. 6.2.1

L_{br} Unbraced length adjacent to the point brace, in. (mm). .... App. 6.2.2

L_c Effective length of member, in. (mm) E2

L_c Effective length of member for buckling about the minor axis, in. (mm) E5

L_c Effective length of built-up member, in. (mm) E6.1

L_{cx} Effective length of member for buckling about x-axis, in. (mm) E4

L_{cy} Effective length of member for buckling about y-axis, in. (mm) E4

L_{cz} Effective length of member for buckling about longitudinal axis, in. (mm) E4

L_{c1} Effective length in the plane of bending, calculated based on the assumption of no lateral translation at the member ends, set equal to the laterally unbraced length of the member unless analysis justifies a smaller value, in. (mm).......... App. 8.2.1

L_{in} Load introduction length, determined in accordance with Section I6.4, in. (mm) I6.3c

L_p Limiting laterally unbraced length for the limit state of yielding, in. (mm) F2.2

L_r Limiting laterally unbraced length for the limit state of inelastic lateral-torsional buckling, in. (mm) F2.2

L_r Nominal roof live load.......... App. 5.4.1

L_v Distance from maximum to zero shear force, in. (mm) G5

L_x, L_y, L_z Laterally unbraced length of the member for each axis, in. (mm) E4

M_A Absolute value of moment at quarter point of the unbraced segment, kip-in. (N-mm) F1

M_B Absolute value of moment at centerline of the unbraced segment, kip-in. (N-mm) F1

M_C Absolute value of moment at three-quarter point of the unbraced segment, kip-in. (N-mm.......... F1

M_c Available flexural strength, ϕM_n or M_n/Ω, determined in accordance with Chapter F, kip-in. (N-mm) H1.1

M_c Available flexural strength, determined in accordance with Section I3, kip-in. (N-mm).......... I5

M_{cr} Elastic lateral-torsional buckling moment, kip-in. (N-mm).......... F10.2

M_{cx} Available lateral-torsional strength for major axis flexure determined in accordance with Chapter F using C_b = 1.0, kip-in. (N-mm) H1.3

M_{cx} Available flexural strength about x-axis for the limit state of tensile rupture of the flange, ϕM_n or M_n/Ω, determined according to Section F13.1, kip-in. (N-mm) H4

M_{lt} First-order moment using LRFD or ASD load combinations, due to lateral translation of the structure only, kip-in. (N-mm).......... App. 8.2

M_{max} Absolute value of maximum moment in the unbraced segment, kip-in. (N-mm) F1

M_n Nominal flexural strength, kip-in. (N-mm).......... F1

M_{nt} First-order moment using LRFD or ASD load combinations, with the structure restrained against lateral translation, kip-in. (N-mm) ... App. 8.2

M_p Plastic moment, kip-in. (N-mm)Table B4.1b

M_p Moment corresponding to plastic stress distribution over the composite cross section, kip-in. (N-mm) I3.4b

M_{pf} Plastic moment of a section composed of the flange and a segment of the web with a depth, d_e, kip-in. (N-mm) G2.3

M_{pm} Smaller of M_{pf} and M_{pst}, kip-in. (N-mm).......... G2.3

M_{pst} Plastic moment of a section composed of the stiffener plus a length of web equal to d_e plus the distance from the inside face of the stiffener to the end of the beam, except that the distance from the inside face of the stiffener to the end of the beam shall not exceed $0.84t_w\sqrt{E/F_y}$ for calculation purposes, kip-in. (N-mm) G2.3

M_r Required second-order flexural strength using LRFD or ASD load combinations, kip-in. (N-mm) .... App. 8.2

M_r Required flexural strength, determined in accordance with Chapter C, using LRFD or ASD load combinations, kip-in. (N-mm) H1.1

M_r Required flexural strength, determined in accordance with Section I1.5, using LRFD or ASD load combinations, kip-in. (N-mm) I5

M_r Required flexural strength of the beam within the panel under consideration using LRFD or ASD load combinations, kip-in. (N-mm) ... . App. 6.3.1a

M_r Largest of the required flexural strengths of the beam within the unbraced lengths adjacent to the point brace using LRFD or ASD load combinations, kip-in. (N-mm)…………………................ App. 6.3.1b

M_{rx} Required flexural strength at the location of the bolt holes, determined in accordance with Chapter C, using LRFD or ASD load combinations, positive for tension in the flange under consideration and negative for compression, kip-in. (N-mm) H4

M_{br} Required flexural strength of the brace, kip-in. (N-mm) .... App. 6.3.2a

M_{ro} Required flexural strength in chord at a joint, on the side of joint with lower compression stress, kip-in. (N-mm).............................. Table K2.1

$M_{r\text{-}ip}$ Required in-plane flexural strength in branch using LRFD or ASD load combinations, kip-in. (N-mm) .... Table K4.1

$M_{r\text{-}op}$ Required out-of-plane flexural strength in branch using LRFD or ASD load combinations, kip-in. (N-mm) .... Table K4.1

M_{rx} Required flexural strength at the location of the bolt holes, determined in accordance with Chapter C, using LRFD or ASD load combinations, positive for tension in the flange under consideration,kip-in. (N-mm)...... H4

M_y Moment at yielding of the extreme fiber, kip-in. (N-mm) ..... Table B4.1b

M_y Yield moment corresponding to yielding of the tension flange and first yield of the compression flange, kip-in. (N-mm)..............................I3.4b

M_y Yield moment about the axis of bending, kip-in. (N-mm).................F9.1

M_y Yield moment calculated using the geometric section modulus, kip-in. (N-mm) F10.2

M_{yc} Yield moment in the compression flange, kip-in. (N-mm) F4.1

M_{yt} Yield moment in the tension flange, kip-in. (N-mm).........................F4.4

M_1' Effective moment at the end of the unbraced length opposite from M_2, kip-in. (N-mm)... App. 1.3.2c

M_1 Smaller moment at end of unbraced length, kip-in. (N-mm) F13.5

M_2 Larger moment at end of unbraced length, kip-in. (N-mm).............F13.5

N_i Notional load applied at level i, kips (N) C2.2b

N_i Additional lateral load, kips (N) .... App. 7.3.2

O_v Overlap connection coefficient K3.1

P_a Required axial strength in chord using ASD load combinations, kips (N) .... Table K2.1

P_{br} Required end and intermediate point brace strength using LRFD or ASD load combinations, kips (N).. App. 6.2.2

P_c Available compressive strength, ϕP_n or P_n/Ω, determined in accordance with Chapter E, kips (N)H1.1

P_c Available tensile strength, ϕP_n or P_n/Ω, determine in accordance with Chapter D, kips (N).. H1.2

P_c Available compressive strength in plane of bending, kips (N) H1.3

P_c Available tensile or compressive strength, ϕP_n or P_n/Ω, determined in accordance with Chapter D or E, kips (N) H3.2

P_c Available axial strength for the limit state of tensile rupture of the net section at the location of bolt holes ϕP_n or P_n/Ω, determined in accordance with Section D2(b), kips (N) H4

P_c Allowable axial strength, determined in accordance with Section I2, kips (N)........ I5

P_c Design axial strength, determined in accordance with Section I2, kips (N)........ I5

P_{cy} Available compressive strength out of the plane of bending, kips (N) H1.3

P_e Elastic critical buckling load determined in accordance with Chapter C or Appendix 7, kips (N) I2.1b

$P_{e\,story}$ Elastic critical buckling strength for the story in the direction of translation being considered, kips (N) .... App 8.2.2

P_{e1} Elastic critical buckling strength of the member in the plane of bending, kips (N) .... App. 8.2.1

P_{lt} First-order axial force using LRFD or ASD load combinations, due to lateral translation of the structure only, kips (N) .... App. 8.2

P_{mf} Total vertical load in columns in the story that are part of moment frames, if any, in the direction of translation being considered, kips (N) .... App. 8.2.2

P_n Nominal axial strength, kips (N) D2

P_n Nominal compressive strength, kips (N) E1

P_{no} Nominal axial compressive strength of zero length, doubly symmetric, axially loaded composite member, kips (N) I2.1b

P_{no} Available compressive strength of axially loaded doubly symmetric filled composite members, kips (N)........ I2.2b

P_{no} Nominal axial compressive strength without consideration of length effects, kips (N) I6.2a

P_{ns} Cross-section compressive strength, kips (N) C2.3

P_{nt} First-order axial force using LRFD and ASD load combinations, with the structure restrained against lateral translation, kips (N)........ App. 8.2

P_p Nominal bearing strength, kips (N) J8

P_r Largest of the required axial strengths of the column within the unbraced lengths adjacent to the point brace, using LRFD or ASD load combinations, kips (N) .... App. 6.2.2

P_r Required axial compressive strength using LRFD or ASD load combinations, kips (N)........ C2.3

P_r Required axial strength of the column within the panel under consideration, using LRFD or ASD load combinations, kips (N) .... App. 6.2.1

P_r Required second-order axial strength using LRFD or ASD load combinations, kips (N)........ App. 8.2

P_r Required compressive strength, determined in accordance with Chapter C, using LRFD or ASD load combinations, kips (N) H1.1

P_r Required tensile strength, determined in accordance with Chapter C, using LRFD or ASD load combinations, kips (N)........ H1.2

P_r Required axial strength, determined in accordance with Chapter C, using LRFD or ASD load combinations, kips (N) H3.2

P_r Required axial strength of the member at the location of the bolt holes, determined in accordance with Chapter C, using LRFD or ASD load combinations, positive in tension and negative in compression, kips (N) H4

P_r Required axial strength, determined in accordance with Section I1.5, using LRFD or ASD load combinations, kips (N)........ I5

P_r Required external force applied to the composite member, kips (N)........ I6.2a

P_r	Required axial strength using LRFD or ASD load combinations, kips (N)	J10.6
P_{ro}	Required axial strength in chord at a joint, on the side of joint with lower compression stress, kips (N)	Table K2.1
P_{story}	Total vertical load supported by the story using LRFD or ASD load combinations, as applicable, including loads in columns that are not part of the lateral force-resisting system, kips (N)	App. 8.2.2
P_u	Required axial strength in chord using LRFD load combinations, kips (N)	Table K2.1
P_u	Required axial strength in compression using LRFD load combinations, kips (N)	App. 1.3.2b
P_y	Axial yield strength of the column, kips (N)	J10.6
Q_{ct}	Available tensile strength, determined in accordance with Section I8.3b, kips (N)	I8.3c
Q_{cv}	Available shear strength, determined in accordance with Section I8.3a, kips (N)	I8.3c
Q_f	Chord-stress interaction parameter	J10.3
Q_g	Gapped truss joint parameter accounting for geometric effects	Table K3.1
Q_n	Nominal shear strength of one steel headed stud or steel channel anchor, kips (N)	I3.2d.1
Q_{nt}	Nominal tensile strength of steel headed stud anchor, kips (N)	I8.3b
Q_{nv}	Nominal shear strength of steel headed stud anchor, kips (N)	I8.3a
Q_{rt}	Required tensile strength, kips (N)	I8.3b
Q_{rv}	Required shear strength, kips (N)	I8.3c
R	Radius of joint surface, in. (mm)	Table J2.2
R_a	Required strength using ASD load combinations	B3.2
R_{FIL}	Reduction factor for joints using a pair of transverse fillet welds only	App. 3.3
R_g	Coefficient to account for group effect	I8.2a
R_M	Coefficient to account for influence of P-δ on P-Δ	App. 8.2.2
R_n	Nominal strength	B3.1
R_n	Nominal bond strength, kips (N)	I6.3c
R_n	Nominal slip resistance, kips (N)	J1.8
R_n	Nominal strength of the applicable force transfer mechanism, kips (N)	I6.3
R_n	Nominal strength of the connected material, kips (N)	J3.10
R_{nwl}	Total nominal strength of longitudinally loaded fillet welds, as determined in accordance with Table J2.5, kips (N)	J2.4
R_{nwt}	Total nominal strength of transversely loaded fillet welds, as determined in accordance with Table J2.5 without the increase in Section J2.4(b), kips (N) J2.4	
R_p	Position effect factor for shear studs	I8.2a
R_{pc}	Web plastification factor, determined in accordance with Section F4.2(c)(6)	F4.1
R_{pg}	Bending strength reduction factor	F5.2
R_{PJP}	Reduction factor for reinforced or nonreinforced transverse partial-joint-penetration (PJP) groove welds	App. 3.3
R_{pt}	Web plastification factor corresponding to the tension flange yielding limit state	F4.4
R_u	Required strength using LRFD load combinations	B3.1
S	Elastic section modulus about the axis of bending, in.^{3} (mm^3)	F7.2
S	Largest clear spacing of the ties, in. (mm)	I1.6b
S	Nominal snow load, kips (N)	App. 4.1.4
S_c	Elastic section modulus, in.^{3} (mm^3)	F9.4

S_c	Elastic section modulus to the toe in compression relative to the axis of bending, in.^{3} (mm^3)	F10.3
S_e	Effective section modulus determined with the effective width of the compression flange, in.^{3} (mm^3)	F7.2
S_{ip}	Effective elastic section modulus of welds for in-plane bending, in.^{3} (mm^3)	K5
S_{min}	Minimum elastic section modulus relative to the axis of bending, in.^{3} (mm^3)	F12
S_x	Elastic section modulus taken about the x-axis, in.^{3} (mm^3)	F2.2
S_x	Minimum elastic section modulus taken about the x-axis, in.^{3} (mm^3)	F13.1
S_{op}	Effective elastic section modulus of welds for out-of-plane bending, in.^{3} (mm^3)	K5
S_{xc}, S_{xt}	Elastic section modulus referred to compression and tension flanges, respectively, in.^{3} (mm^3)	Table B4.1b
S_y	Elastic section modulus taken about the y-axis, in.^{3} (mm^3)	F6.1
T	Elevated temperature of steel due to unintended fire exposure, °F (°C)	App. 4.2.4d
T_a	Required tension force using ASD load combinations, kips (kN)	J3.9
T_b	Minimum fastener tension given in Table J3.1 or J3.1M, kips (kN)	J3.8
T_c	Available torsional strength, ϕT_n or T_n/Ω, determined in accordance with Section H3.1, kip-in. (N-mm)	H3.2
T_n	Nominal torsional strength, kip-in. (N-mm)	H3.1
T_r	Required torsional strength, determined in accordance with Chapter C, using LRFD or ASD load combinations, kip-in. (N-mm)	H3.2
T_u	Required tension force using LRFD load combinations, kips (kN)	J3.9
U	Shear lag factor	D3
U	Utilization ratio	Table K2.1
U_{bs}	Reduction coefficient, used in calculating block shear rupture strength	J4.3
U_p	Stress index for primary members	App. 2.2
V'	Nominal shear force between the steel beam and the concrete slab transferred by steel anchors, kips (N)	I3.2d
V_{br}	Required shear strength of the bracing system in the direction perpendicular to the longitudinal axis of the column, kips (N)	App. 6.2.1
V_c	Available shear strength, ϕV_n or V_n/Ω, determined in accordance with Chapter G, kips (N)	H3.2
V_{c1}	Available shear strength calculated with V_n as defined in Section G2.1 or G2.2. as applicable, kips (N)	G2.4
V_{c2}	Available shear buckling strength, kips (N)	G2.4
V_n	Nominal shear strength, kips (N)	G1
V_r	Required shear strength in the panel being considered, kips (N)	G2.4
V_r	Required shear strength, determined in accordance with Chapter C, using LRFD or ASD load combinations, kips (N)	H3.2
V_r'	Required longitudinal shear force to be transferred to the steel or concrete, kips (N)	I6.1
Y_i	Gravity load applied at level i from the LRFD load combination or ASD load combination, as applicable, kips (N)	C2.2b
Z	Plastic section modulus taken about the axis of bending, in.^{3} (mm^3)	F7.1
Z_b	Plastic section modulus of branch taken about the axis of bending, in.^{3} (mm^3)	K4.1
Z_x	Plastic section modulus taken about the x-axis, in.^{3} (mm^3)	Table B4.1b
Z_y	Plastic section modulus taken about the y-axis, in.^{3} (mm^3)	F6.1

a	Clear distance between transverse stiffeners, in. (mm)	F13.2
a	Distance between connectors, in. (mm)	E6.1
a	Shortest distance from edge of pin hole to edge of member measured parallel to the direction of force, in. (mm)	D5.1
a	Half the length of the nonwelded root face in the direction of the thickness of the tension-loaded plate, in. (mm)	App. 3.3
a'	Weld length along both edges of the cover plate termination to the beam or girder, in. (mm)	F13.3
a_w	Ratio of two times the web area in compression due to application of major axis bending moment alone to the area of the compression flange components	F4.2
b	Full width of leg in compression, in. (mm)	F10.3
b	Largest clear distance between rows of steel anchors or ties, in. (mm)	I1.6a
b	Width of compression element as shown in Table B4.1, in. (mm)	B4.1
b	Width of the element, in. (mm)	E7.1
b	Width of compression flange as defined in Section B4.1b, in. (mm)	F7.2
b	Width of the leg resisting the shear force or depth of tee stem, in. (mm)	G3
b	Width of leg, in. (mm)	F10.2
b_{cf}	Width of column flange, in. (mm)	J10.6
b_e	Effectivewidth, in. (mm)	E7.1
b_e	Effective edge distance for calculation of tensile rupture strength of pin-connected member, in. (mm)	D5.1
b_f	Width of flange, in. (mm)	B4.1
b_{fc}	Width of compression flange, in. (mm)	F4.2
b_{ft}	Width of tension flange, in. (mm)	G2.2
b_l	Length of longer leg of angle, in. (mm)	E5
b_p	Smaller of the dimension a and h, in. (mm)	G2.4
b_s	Length of shorter leg of angle, in. (mm)	E5
b_s	Stiffener width for one-sided stiffeners; twice the individual stiffener width for pairs of stiffeners, in. (mm)	App. 6.3.2a
c	Distance from the neutral axis to the extreme compressive fibers, in. (mm)	App. 6.3.2a
c_1	Effective width imperfection adjustment factor determined from Table E7.1	E7.1
d	Depth of section from which the tee was cut, in. (mm)	Table D3.1
d	Depth of tee or width of web leg in tension, in. (mm)	F9.2
d	Depth of tee or width of web leg in compression, in. (mm)	F9.2
d	Nominal diameter of fastener, in. (mm)	J3.3
d	Full depth of thesection, in. (mm)	B4.1a
d	Depth of rectangular bar, in. (mm)	F11.1
d	Diameter, in. (mm)	J7
d	Diameter of pin, in. (mm)	D5.1
d_b	Depth of beam, in. (mm)	J10.6
d_b	Nominal diameter (body or shank diameter), in. (mm)	App. 3.4
d_c	Depth of column, in. (mm)	J10.6
d_e	Effective width for tees, in. (mm)	E7.1
d_h	Nominal hole diameter plus 1/16 in. (2 mm), in. (mm)	J4.3
d_{sa}	Diameter of steel headed stud anchor, in. (mm)	I8.1
d_{tie}	Effective diameter of the tie bar, in. (mm)	I1.6b
e	Eccentricity in a truss connection, positive being away from the branches, in. (mm)	K3.1
$e_{mid\text{-}ht}$	Distance from the edge of steel headed stud anchor shank to the steel deck web, in. (mm)	I8.2a

f_c'	Specified compressive strength of concrete, ksi (MPa)	I1.3
f_{ra}	Required axial stress at the point of consideration, determined in accordance with Chapter C, using LRFD or ASD load combinations, ksi (MPa)	H2
f_{rbw}, f_{rbz}	Required flexural stress at the point of consideration, determined in accordance with Chapter C, using LRFD or ASD load combinations, ksi (MPa)	H2
f_{rv}	Required shear stress using LRFD or ASD load combinations, ksi (MPa)	J3.7
g	Transverse center-to-center spacing (gage) between fastener gage lines, in. (mm)	B4.3b
g	Gap between toes of branch members in a gapped K-connection, neglecting the welds, in. (mm)	K3.1
h	Width of compression element as shown in Table B4.1, in. (mm)	B4.1
h	Depth of web, as defined in Section B4.1b, in. (mm)	F7.3
h	Clear distance between flanges less the fillet at each flange, in. (mm)	G2.1
h	For built-up welded sections, the clear distance between flanges, in. (mm)	G2.1
h	For built-up bolted sections, the distance between fastener lines, in. (mm)	G2.1
h	Width resisting the shear force, taken as the clear distance between the flanges less the inside corner radius on each side for HSS or the clear distance between flanges for box sections, in. (mm)	G4
h	Flat width of longer side, as defined in Section B4.1b(d), in. (mm)	H3.1
h_c	Twice the distance from the center of gravity to the following: the inside face of the compression flange less the fillet or corner radius, for rolled shapes; the nearest line of fasteners at the compression flange or the inside faces of the compression flange when welds are used, for built-up sections, in. (mm)	B4.1
h_e	Effective width for webs, in. (mm)	E7
h_f	Factor for fillers	J3.8
h_o	Distance between flange centroids, in. (mm)	E4
h_p	Twice the distance from the plastic neutral axis to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used, in. (mm)	B4.1b
k	Distance from outer face of flange to the web toe of fillet, in. (mm)	J10.2
k_c	Coefficient for slender unstiffened elements	Table B4.1
k_{sc}	Slip-critical combined tension and shear coefficient	J3.9
k_v	Web plate shear buckling coefficient	G2.1
l	Actual length of end-loaded weld, in. (mm)	J2.2
l	Length of connection, in. (mm)	Table D3.1
l_b	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)	K2.1
l_b	Length of bearing, in. (mm)	J7
l_a	Length of channel anchor, in. (mm)	I8.2b
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)	J3.10
l_e	Total effective weld length of groove and fillet welds to HSS for weld strength calculations, in. (mm)	K5
l_{end}	Distance from the near side of the connecting branch or plate to end of chord, in. (mm)	K1.1

l_{ov}	Overlap length measured along the connecting face of the chord beneath the two branches, in. (mm)	K3.1
l_p	Projected length of the overlapping branch on the chord, in. (mm)	K3.1
n	Number of braced points within the span	App. 6.3.2a
n	Threads per inch (per mm)	App. 3.4
n_b	Number of bolts carrying the applied tension	J3.9
n_s	Number of slip planes required to permit the connection to slip	J3.8
n_{SR}	Number of stress range fluctuations in design life	App. 3.3
p	Pitch, in. per thread (mm per thread)	App. 3.4
p_b	Perimeter of the steel-concrete bond interface within the composite cross section, in. (mm)	I6.3c
r	Radius of gyration, in. (mm)	E2
r	Retention factor depending on bottom flange temperature	App. 4.2.4d
r_a	Radius of gyration about the geometric axis parallel to the connected leg, in. (mm)	E5
r_i	Least radius of gyration of individual component, in. (mm)	E6.1
$\overline{r}_o$	Polar radius of gyration about the shear center, in. (mm)	E4
r_t	Effective radius of gyration for lateral-torsional buckling. For I-shapes with a channel cap or a cover plate attached to the compression flange, radius of gyration of the flange components in flexural compression plus one-third of the web area in compression due to application of major axis bending moment alone, in. (mm)	F4.2
r_x	Radius of gyration about the x-axis, in. (mm)	E4
r_y	Radius of gyration about y-axis, in. (mm)	E4
r_z	Radius of gyration about the minor principal axis, in. (mm)	E5
s	Longitudinal center-to-center spacing (pitch) of any two consecutive holes, in. (mm)	B4.3b
t	Distance from the neutral axis to the extreme tensile fibers, in. (mm)	App. 6.3.2a
t	Plate thickness, in. (mm)	I1.6a
t	Thickness of wall, in. (mm)	E7.2
t	Thickness of angle leg, in. (mm)	F10.2
t	Width of rectangular bar parallel to axis of bending, in. (mm)	F11.1
t	Thickness of connected material, in. (mm)	J3.10
t	Thickness of plate, in. (mm)	D5.1
t	Total thickness of fillers, in. (mm)	J5.2
t	Design wall thickness of HSS member, in. (mm)	B4. 2
t	Design wall thickness of HSS main member, in. (mm)	K1.1
t	Thickness of angle leg or tee stem, in. (mm)	G3
t_b	Design wall thickness of HSS branch member or thickness of plate, in. (mm)	K1.1
t_{bi}	Thickness of overlapping branch, in. (mm)	Table K3.2
t_{bj}	Thickness of overlapped branch, in. (mm)	Table K3.2
t_{cf}	Thickness of column flange, in. (mm)	J10.6
t_f	Thickness of flange, in. (mm)	F3.2
t_f	Thickness of the loaded flange, in. (mm)	J10.1
t_f	Thickness of flange of channel anchor, in. (mm)	I8.2b
t_{fc}	Thickness of compression flange, in. (mm)	F4.2
t_p	Thickness of tension loaded plate, in. (mm)	App. 3.3
t_{sc}	Thickness of composite plate shear wall, in. (mm)	I1.6b
t_{st}	Thickness of web stiffener, in. (mm)	App. 6.3.2a
t_w	Thickness of web, in. (mm)	F4.2
t_w	Smallest effective weld throat thickness around the perimeter of branch or plate, in. (mm)	K5
t_w	Thickness of channel anchor web, in. (mm)	I8.2b

t_w — Thickness of column web, in. (mm) ... J10.6
w — Width of cover plate, in. (mm) ... F13.3
w — Weld size, in. (mm) ... J2.2b
w — Subscript relating symbol to major principal axis bending ... H2
w — Width of plate, in. (mm) ... Table D3.1
w — Leg size of the reinforcing or contouring fillet, if any, in the direction of the thickness of the tension-loaded plate, in. (mm) ... App. 3.3
w_c — Weight of concrete per unit volume ($90 \le w_c \le 155$ lb/ft^3 or $1\,500 \le w_c \le 2\,500$ kg/m^3) ... I2.1b
w_r — Average width of concrete rib or haunch, in. (mm) ... I3.2c
x — Subscript relating symbol to major axis bending ... H1.1
x_o, y_o — Coordinates of the shear center with respect to the centroid, in. (mm) ... E4
$\bar{x}$ — Eccentricity of connection, in. (mm) ... Table D3.1
y — Subscript relating symbol to minor axis bending ... H1.1
y_a — Bracing offset distance along y-axis, in. (mm) ... E4
z — Subscript relating symbol to minor principal axis bending ... H2
β — Length reduction factor given by Equation J2-1 ... J2.2b
β — Width ratio; the ratio of branch diameter to chord diameter for round HSS; the ratio of overall branch width to chord width for rectangular HSS ... K1.1
β_T — Overall brace system required stiffness, kip-in./rad (N-mm/rad) ... App. 6.3.2a
β_{br} — Required shear stiffness of the bracing system, kip/in. (N/mm) ... App. 6.2.1
β_{br} — Required flexural stiffness of the brace, kip/in. (N/mm) ... App. 6.3.2a
β_{eff} — Effective width ratio; the sum of the perimeters of the two branch members in a K-connection divided by eight times the chord width ... K1.1
β_{eop} — Effective outside punching parameter ... Table K3.2
β_{sec} — Web distortional stiffness, including the effect of web transverse stiffeners, if any, kip-in./rad (N-mm/rad) ... App. 6.3.2a
β_w — Section property for single angles about major principal axis, in. (mm) ... F10.2
Δ — First-order interstory drift due to the LRFD or ASD load combinations, in. (mm) ... App. 7.3.2
Δ_H — First-order interstory drift, in the direction of translation being considered, due to lateral forces, in. (mm) ... App. 8.2.2
γ — Chord slenderness ratio; the ratio of one-half the diameter to the wall thickness for round HSS; the ratio of one-half the width to wall thickness for rectangular HSS ... K3.1
ζ — Gap ratio; the ratio of the gap between the branches of a gapped K-connection to the width of the chord for rectangular HSS ... K3.1
η — 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 ... K3.1
λ — Width-to-thickness ratio for the element as defined in Section B4.1 ... E7.1
λ_{pf} — Limiting width-to-thickness ratio for compact flange, as defined in Table B4.1b F3.2
λ_{pw} — Limiting width-to-thickness ratio for compact web, as defined in Table B4.1b ... F4.2
λ_r — Limiting width-to-thickness ratio as defined in Table B4.1a ... E7.1

λ_{rf}	Limiting width-to-thickness ratio for noncompact flange, as defined in Table B4.1b	F3.2
λ_{rw}	Limiting width-to-thickness ratio for noncompact web, as defined in Table B4.1b	F4.2
μ	Mean slip coefficient for Class A or B surfaces, as applicable, or as established by tests	J3.8
ϕ	Resistance factor	B3.1
ϕ_B	Resistance factor for bearing on concrete	I6.3a
ϕ_b	Resistance factor for flexure	I5
ϕ_c	Resistance factor for compression	I5
ϕ_c	Resistance factor for axially loaded composite columns	I2.1b
ϕ_d	Resistance factor for direct bond interaction	I6.3c
ϕ_{sf}	Resistance factor for shear on the failure path	D5.1
ϕ_T	Resistance factor for torsion	H3.1
ϕ_t	Resistance factor for tension	H1.2
ϕ_t	Resistance factor for tensile rupture	H4
ϕ_t	Resistance factor for steel headed stud anchor in tension	I8.3b
ϕ_v	Resistance factor for shear	G1
ϕ_v	Resistance factor for steel headed stud anchor in shear	I8.3a
Ω	Safety factor	B3.2
Ω_B	Safety factor for bearing on concrete	I6.3a
Ω_b	Safety factor for flexure	I5
Ω_c	Safety factor for compression	I5
Ω_c	Safety factor for axially loaded composite columns	I2.1b
Ω_d	Safety factor for direct bond interaction	I6.3c
Ω_t	Safety factor for steel headed stud anchor in tension	I8.3b
Ω_{sf}	Safety factor for shear on the failure path	D5.1
Ω_T	Safety factor for torsion	H3.1
Ω_t	Safety factor for tension	H1.2
Ω_t	Safety factor for tensile rupture	H4
Ω_v	Safety factor for shear	G1
Ω_v	Safety factor for steel headed stud anchor in shear	I8.3a
ρ_w	Maximum shear ratio within the web panels on each side of the transverse stiffener	G2.4
ρ_{sr}	Minimum reinforcement ratio for longitudinal reinforcing	I2.1
θ	Angle between the line of action of the required force and the weld longitudinal axis, degrees	J2.4
θ	Acute angle between the branch and chord, degrees	K3.1

GLOSSARY

Notes:

(1) Terms designated with † are common AISI-AISC terms that are coordinated between the two standards development organizations.

(2) Terms designated with * are usually qualified by the type of load effect, for example, nominal tensile strength, available compressive strength, and design flexural strength.

(3) Terms designated with ** are usually qualified by the type of component, for example, web local buckling, and flange local bending.

Active fire protection. Building materials and systems that are activated by a fire to mitigate adverse effects or to notify people to take action to mitigate adverse effects.

*Allowable strength**†. Nominal strength divided by the safety factor, R_n/Ω.

*Allowable stress**. Allowable strength divided by the applicable section property, such as section modulus or cross-sectional area.

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.

Applicable building code†. Building code under which the structure is designed.

Approval documents. The structural steel shop drawings, erection drawings, and embedment drawings, or where the parties have agreed in the contract documents to provide digital model(s), the fabrication and erection models. A combination of drawings and digital models also may be provided.

ASD (allowable strength design)†. Method of proportioning structural components such that the allowable strength equals or exceeds the required strength of the component under the action of the ASD load combinations.

ASD load combination†. Load combination in the applicable building code intended for allowable strength design (allowable stress design).

Authority having jurisdiction (AHJ). Organization, political subdivision, office or individual charged with the responsibility of administering and enforcing the provisions of this *Specification.*

*Available strength**†. Design strength or allowable strength, as applicable.

*Available stress**. Design stress or allowable stress, as applicable.

Average rib width. In a formed steel deck, average width of the rib of a corrugation.

Beam. Nominally horizontal structural member that has the primary function of resisting bending moments.

Beam-column. Structural member that resists both axial force and bending moment.

Bearing (local compressive yielding)†. Limit state of local compressive yielding due to the action of a member bearing against another member or surface.

Bearing-type connection. Bolted connection where shear forces are transmitted by the bolt bearing against the connection elements.

Block shear rupture†. In a connection, limit state of tension rupture along one path and shear yielding or shear rupture along another path.

Bolting assembly. An assembly of bolting components that is installed as a unit.

Bolting component. Bolt, nut, washer, direct tension indicator or other element used as a part of a bolting assembly.

Box section. Square or rectangular doubly symmetric member made with four plates welded together at the corners such that it behaves as a single member.

Braced frame†. Essentially vertical truss system that provides resistance to lateral forces and provides stability for the structural system.

Bracing. Member or system that provides stiffness and strength to limit the out-of-plane movement of another member at a brace point.

Branch member. In an HSS connection, member that terminates at a chord member or main member.

Buckling†. Limit state of sudden change in the geometry of a structure or any of its elements under a critical loading condition.

Buckling strength. Strength for instability limit states.

Built-up member, cross section, section, shape. Member, cross section, section or shape fabricated from structural steel elements that are welded or bolted together.

Camber. Curvature fabricated into a beam or truss so as to compensate for deflection induced by loads.

Charpy V-notch impact test. Standard dynamic test measuring notch toughness of a specimen.

Chord member. In an HSS connection, primary member that extends through a truss connection.

Cladding. Exterior covering of structure.

Cold-formed steel structural member†. Shape manufactured by press-braking blanks sheared from sheets, cut lengths of coils or plates, or by roll forming cold- or hot-rolled coils or sheets; both forming operations being performed at ambient room temperature, that is, without manifest addition of heat such as would be required for hot forming.

Collector. Also known as drag strut; member that serves to transfer loads between floor diaphragms and the members of the lateral force-resisting system.

Column. Nominally vertical structural member that has the primary function of resisting axial compressive force.

Column base. Assemblage of structural shapes, plates, connectors, bolts and rods at the base of a column used to transmit forces between the steel superstructure and the foundation.

Combined method. Pretensioning procedure incorporating the application of a prescribed initial torque or tension, followed by the application of a prescribed relative rotation between the bolt and nut.

Compact section. Section that can reach the plastic moment before local buckling occurs as defined by the element width to thickness ratio less than or equal to λ_p.

Compartmentation. Enclosure of a building space with elements that have a specific fire endurance.

Complete-joint-penetration (CJP) groove weld. Groove weld in which weld metal extends through the joint thickness, except as permitted for HSS connections.

Composite. Condition in which steel and concrete elements and members work as a unit in the distribution of internal forces.

Composite beam. Structural steel beam in contact with and acting compositely with a reinforced concrete slab.

Composite component. Member, connecting element or assemblage in which steel and concrete elements work as a unit in the distribution of internal forces, with the exception of the special case of composite beams where steel anchors are embedded in a solid concrete slab or in a slab cast on formed steel deck.

Concrete breakout surface. The surface delineating a volume of concrete surrounding a steel headed stud anchor that separates from the remaining concrete.

Concrete crushing. Limit state of compressive failure in concrete having reached the ultimate strain.

Concrete haunch. In a composite floor system constructed using a formed steel deck, the section of solid concrete that results from stopping the deck on each side of the girder.

Concrete-encased beam. Beam totally encased in concrete cast integrally with the slab.

Connection†. Combination of structural elements and joints used to transmit forces between two or more members.

Construction documents. Written, graphic and pictorial documents prepared or assembled for describing the design (including the structural system), location and physical characteristics of the elements of a building necessary to obtain a building permit and construct a building.

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 documents, the specifications, and the contract.

Cope. Cutout made in a structural member to remove a flange and conform to the shape of an intersecting member.

Cover plate. Plate welded or bolted to the flange of a member to increase cross-sectional area, section modulus or moment of inertia.

Cross connection. HSS connection in which forces in branch members or connecting elements transverse to the main member are primarily equilibrated by forces in other branch members or connecting elements on the opposite side of the main member.

Design. The process of establishing the physical and other properties of a structure for the purpose of achieving the desired strength, serviceability, durability, constructability, economy and other desired characteristics. Design for strength, as used in this *Specification*, includes analysis to determine required strength and proportioning to have adequate available strength.

Design-basis fire. Set of conditions that define the development of a fire and the spread of combustion products throughout a building or portion thereof.

Design documents. The graphic and pictorial portions of the contract documents showing the design, location, and dimensions of work. These documents generally include, but are not necessarily limited to, plans, elevations, sections, details, schedules, diagrams, and notes. Where the parties have agreed in the contract documents to provide digital model(s), a dimensionally accurate 3D digital model of the structure that conveys the structural steel requirements given in AISC *Code of Standard Practice for Steel Buildings and Bridges*, Section 3.1. A combination of drawings and digital models also may be provided.

Design load†. Applied load determined in accordance with either LRFD load combinations or ASD load combinations, as applicable.

*Design strength**†. Resistance factor multiplied by the nominal strength, ϕR_n.

Design wall thickness. HSS wall thickness assumed in the determination of section properties.

Diagonal stiffener. Web stiffener at column panel zone oriented diagonally to the flanges, on one or both sides of the web.

Diaphragm†. Roof, floor or other membrane or bracing system that transfers in-plane forces to the lateral force-resisting system.

Direct bond interaction. In a composite section, mechanism by which force is transferred between steel and concrete by bond stress.

Distortional failure. Limit state of an HSS truss connection based on distortion of a rectangular HSS chord member into a rhomboidal shape.

Distortional stiffness. Out-of-plane flexural stiffness of web.

Double curvature. Deformed shape of a beam with one or more inflection points within the span.

Double-concentrated forces. Two equal and opposite forces applied normal to the same flange, forming a couple.

Doubler. Plate added to, and parallel with, a beam or column web to increase strength at locations of concentrated forces.

Drift. Lateral deflection of structure.

Effective length factor, K. Ratio between the effective length and the unbraced length of the member.

Effective length. Length of an otherwise identical compression member with the same strength when analyzed with simple end conditions.

Effective net area. Net area modified to account for the effect of shear lag.

Effective section modulus. Section modulus reduced to account for buckling of slender compression elements.

Effective width. Reduced width of a plate or slab with an assumed uniform stress distribution which produces the same effect on the behavior of a structural member as the actual plate or slab width with its nonuniform stress distribution.

Elastic analysis. Structural analysis based on the assumption that the structure returns to its original geometry on removal of the load.

Elevated temperatures. Heating conditions experienced by building elements or structures as a result of fire which are in excess of the anticipated ambient conditions.

Encased composite member. Composite member consisting of a structural concrete member and one or more embedded steel shapes.

End panel. Web panel with an adjacent panel on one side only.

End return. Length of fillet weld that continues around a corner in the same plane.

Engineer of record. Licensed professional responsible for sealing the design documents and specifications.

Erection documents. The field-installation or member-placement drawings that are prepared by the fabricator to show the location and attachment of the individual structural steel shipping pieces. Where the parties have agreed in the contract documents to provide digital model(s), a dimensionally accurate 3D digital model produced to convey the information necessary to erect the structural steel, which may be the same digital model as the fabrication model. A combination of drawings and digital models also may be provided.

Expansion rocker. Support with curved surface on which a member bears that is able to tilt to accommodate expansion.

Expansion roller. Round steel bar on which a member bears that is able to roll to accommodate expansion.

Eyebar. Pin-connected tension member of uniform thickness, with forged or thermally cut head of greater width than the body, proportioned to provide approximately equal strength in the head and body.

Fabrication documents. The shop drawings of the individual structural steel shipping pieces that are to be produced in the fabrication shop. Where the parties have agreed in the contract documents to provide digital model(s), a dimensionally accurate 3D digital model produced to convey the information necessary to fabricate the structural steel, which may be the same digital model as the erection model. A combination of drawings and digital models also may be provided.

Factored load †. Product of a load factor and the nominal load.

Fastener. Generic term for bolts, rivets or other connecting devices.

Fatigue†.Limit state of crack initiation and growth resulting from repeated application of live loads.

Faying surface. Contact surface of connection elements transmitting a shear force.

Filled composite member. Composite member consisting of an HSS or box section filled with structural concrete.

Filler metal. Metal or alloy added in making a welded joint.

Filler. Plate used to build up the thickness of one component.

Fillet weld reinforcement. Fillet welds added to groove welds.

Fillet weld. Weld of generally triangular cross section made between intersecting surfaces of elements.

Finished surface. Surfaces fabricated with a roughness height value measured in accordance with ANSI/ASME B46.1 that is equal to or less than 500.

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 an approved standard fire resistance test, to demonstrate compliance with the applicable building code.

Fire resistance. Property of assemblies that prevents or retards the passage of excessive heat, hot gases or flames under conditions of use and enables the assemblies to continue to perform a stipulated function.

First-order analysis. Structural analysis in which equilibrium conditions are formulated on the undeformed structure; second-order effects are neglect-ed.

Fitted bearing stiffener. Stiffener used at a support or concentrated load that fits tightly against one or both flanges of a beam so as to transmit load through bearing.

Flare bevel groove weld. Weld in a groove formed by a member with a curved surface in contact with a planar member.

Flare V-groove weld. Weld in a groove formed by two members with curved surfaces.

Flashover. Transition to a state of total surface involvement in a fire of combustible materials within an enclosure.

Flat width. Nominal width of rectangular HSS minus twice the outside corner radius. In the absence of knowledge of the corner radius, the flat width is permitted to be taken as the total section width minus three times the thickness.

Flexibility. The ratio of the displacement (or rotation) to the applied force (or moment), which is the inverse of the stiffness.

Flexural buckling†. Buckling mode in which a compression member deflects laterally without twist or change in cross-sectional shape.

Flexural-torsional buckling†. Buckling mode in which a compression member bends and twists simultaneously without change in cross-sectional shape.

Force. Resultant of distribution of stress over a prescribed area.

Formed steel deck. In composite construction, steel cold formed into a decking profile used as a permanent concrete form.

Fully-restrained moment connection. Connection capable of transferring moment with negligible rotation between connected members.

Gage. Transverse center-to-center spacing of fasteners.

Gapped connection. HSS truss connection with a gap or space on the chord face between intersecting branch members.

Geometric axis. Axis parallel to web, flange or angle leg.

Girder filler. In a composite floor system constructed using a formed steel deck, narrow piece of sheet steel used as a fill between the edge of a deck sheet and the flange of a girder.

Girder. See *Beam*.

Gouge. Relatively smooth surface groove or cavity resulting from plastic deformation or removal of material.

Gravity load. Load acting in the downward direction, such as dead and live loads.

Grip (of bolt). Thickness of material through which a bolt passes.

Groove weld. Weld in a groove between connection elements. See also AWS D1.1/D1.1M.

Gusset plate. Plate element connecting truss members or a strut or brace to a beam or column.

Heat flux. Radiant energy per unit surface area.

Heat release rate. Rate at which thermal energy is generated by a burning material.

High-strength bolt. An ASTM F3125 or F3148 bolt, or an alternative design bolt that meets the requirements in RCSC Section 2.13.

Horizontal shear. In a composite beam, force at the interface between steel and concrete surfaces.

HSS (hollow structural section). Square, rectangular or round hollow structural steel section produced in accordance with one of the product specifications in Section A3.1a(b).

Inelastic analysis. Structural analysis that takes into account inelastic material behavior, including plastic analysis.

Initial tension. Minimum bolt tension attained before application of the required rotation when using the combined method to pretension bolting assemblies.

In-plane instability†. Limit state involving buckling in the plane of the frame or the member.

Instability†. Limit state reached in the loading of a structural component, frame or structure in which a slight disturbance in the loads or geometry produces large displacements.

Introduction length. The length along which the required longitudinal shear force is assumed to be transferred into or out of the steel shape in an encased or filled composite column.

Issuing entity. Any party on a project that issues structural design documents and specifications provided by the engineer of record.

Issued for construction. The engineer of record’s designation that the design documents are authorized to be used to construct the steel structure depicted in the design documents, and that these design documents incorporate the information that is to be provided per the requirements of Section A4.

Joint†. Area where two or more ends, surfaces, or edges are attached. Categorized by type of fastener or weld used and method of force transfer.

Joint eccentricity. In an HSS truss connection, perpendicular distance from chord member center-of-gravity to intersection of branch member work points.

k-area. The region of the web that extends from the tangent point of the web and the flange-web fillet (AISC *k* dimension) a distance 1½ in. (38 mm) into the web beyond the *k* dimension.

K-connection. HSS connection in which forces in branch members or connecting elements transverse to the main member are primarily equilibriated by forces in other branch members or connecting elements on the same side of the main member.

Lacing. Plate, angle or other steel shape, in a lattice configuration, that connects two steel shapes together.

Lap joint. Joint between two overlapping connection elements in parallel planes.

Lateral bracing. Member or system that is designed to inhibit lateral buckling or lateral-torsional buckling of structural members.

Lateral force-resisting system. Structural system designed to resist lateral loads and provide stability for the structure as a whole.

Lateral load. Load acting in a lateral direction, such as wind or earthquake effects.

Lateral-torsional buckling†. Buckling mode of a flexural member involving deflection out of the plane of bending occurring simultaneously with twist about the shear center of the cross section.

Leaning column. Column designed to carry gravity loads only, with connections that are not intended to provide resistance to lateral loads.

Length effects. Consideration of the reduction in strength of a member based on its unbraced length.

Lightweight concrete. Structural concrete with an equilibrium density of 115 lb/ft^3 (1 840 kg/m^3) or less, as determined by ASTM C567.

Limit state†. Condition in which a structure or component becomes unfit for service and is judged either to be no longer useful for its intended function (serviceability limit state) or to have reached its ultimate load-carrying capacity (strength limit state).

Load†. *Force* or other action that results from the weight of building materials, occupants and their possessions, environmental effects, differential movement, or restrained dimensional changes.

Load effect†. Forces, stresses and deformations produced in a structural component by the applied loads.

Load factor. Factor that accounts for deviations of the nominal load from the actual load, for uncertainties in the analysis that transforms the load into a load effect and for the probability that more than one extreme load will occur simultaneously.

Load transfer region. Region of a composite member over which force is directly applied to the member, such as the depth of a connection plate.

*Local bending*** †. Limit state of large deformation of a flange under a concentrated transverse force.

*Local buckling***. Limit state of buckling of a compression element within a cross section.

*Local yielding***†. Yielding that occurs in a local area of an element.

LRFD (load and resistance factor design)†. Method of proportioning structural components such that the design strength equals or exceeds the required strength of the component under the action of the LRFD load combinations.

LRFD load combination†. Load combination in the applicable building code intended for strength design (load and resistance factor design).

Main member. In an HSS connection, chord member, column or other HSS member to which branch members or other connecting elements are attached.

Member imperfection. Initial displacement of points along the length of individual members (between points of intersection of members) from their nominal locations, such as the out-of-straightness of members due to manufacturing and fabrication.

Mill scale. Oxide surface coating on steel formed by the hot rolling process.

Moment connection. Connection that transmits bending moment between connected members.

Moment frame†. Framing system that provides resistance to lateral loads and provides stability to the structural system, primarily by shear and flexure of the framing members and their connections.

Negative flexural strength. Flexural strength of a composite beam in regions with tension due to flexure on the top surface.

Net area. Gross area reduced to account for removed material.

Nominal dimension. Designated or theoretical dimension, as in tables of section properties.

Nominal load†. Magnitude of the load specified by the applicable building code.

Nominal rib height. In a formed steel deck, height of deck measured from the underside of the lowest point to the top of the highest point.

*Nominal strength**†. Strength of a structure or component (without the resistance factor or safety factor applied) to resist load effects, as determined in accordance with this specification.

Noncompact section. Section that is not able to reach the plastic moment before inelastic local buckling occurs as defined by element width to thickness ratio greater than λ_p and less than or equal to λ_r.

Nondestructive testing. Inspection procedure wherein no material is destroyed and the integrity of the material or component is not affected.

Notch toughness. Energy absorbed at a specified temperature as measured in the Charpy V-notch impact test.

Notional load. Virtual load applied in a structural analysis to account for destabilizing effects that are not otherwise accounted for in the design provisions.

Out-of-plane buckling†. Limit state of a beam, column or beam-column involving lateral or lateral-torsional buckling.

Overlapped connection. HSS truss connection in which intersecting branch members overlap.

Panel brace. Brace that controls the relative movement of two adjacent brace points along the length of a beam or column or the relative lateral displacement of two stories in a frame (see *point brace*).

Panel zone. Web area of beam-to-column connection delineated by the extension of beam and column flanges through the connection, transmitting moment through a shear panel.

Partial-joint-penetration (PJP) groove weld. Groove weld in which the penetration is intentionally less than the complete thickness of the connected element.

Partially restrained moment connection. Connection capable of transferring moment with rotation between connected members that is not negligible.

Percent elongation. Measure of ductility, determined in a tensile test as the maximum elongation of the gage length divided by the original gage length expressed as a percentage.

Pipe. See *HSS*.

Pitch. Longitudinal center-to-center spacing of fasteners. Center-to-center spacing of bolt threads along axis of bolt.

Plastic analysis. Structural analysis based on the assumption of rigid-plastic behavior, that is, that equilibrium is satisfied and the stress is at or below the yield stress throughout the structure.

Plastic hinge. Fully yielded zone that forms in a structural member when the plastic moment is attained.

Plastic moment. Theoretical resisting moment developed within a fully yielded cross section.

Plastic stress distribution method. In a composite member, method for determining stresses assuming that the steel section and the concrete in the cross section are fully plastic.

Plastification. In an HSS connection, limit state based on an out-of-plane flexural yield line mechanism in the chord at a branch member connection.

Plate girder. Built-up beam.

Plug weld. Weld made in a circular hole in one element of a joint fusing that element to another element.

Point brace. Brace that prevents lateral movement or twist independently of other braces at adjacent brace points (see *panel brace*).

Ponding. Retention of water due solely to the deflection of flat roof framing.

Positive flexural strength. Flexural strength of a composite beam in regions with compression due to flexure on the top surface.

Pretensioned bolt. Bolt tightened to the specified minimum pretension.

Pretensioned joint. Joint with high-strength bolts tightened to the specified minimum pretension.

Properly developed. Reinforcing bars detailed to yield in a ductile manner before crushing of the concrete occurs. Bars meeting the provisions of ACI 318, insofar as development length, spacing and cover are deemed to be properly developed.

Prying action. Amplification of the tension force in a bolt caused by leverage between the point of applied load, the bolt, and the reaction of the connected elements.

Punching load. In an HSS connection, component of branch member force perpendicular to a chord.

P-δ effect. Effect of loads acting on the deflected shape of a member between joints or nodes.

P-Δ effect. Effect of loads acting on the displaced location of joints or nodes in a structure. In tiered building structures, this is the effect of loads acting on the laterally displaced location of floors and roofs.

Quality assurance. Monitoring and inspection tasks to ensure that the material provided and work performed by the fabricator and erector meet the requirements of the approved construction documents and referenced standards. Quality assurance includes those tasks designated "special inspection" by the applicable building code.

Quality assurance inspector (QAI). Individual designated to provide quality assurance inspection for the work being performed.

Quality assurance plan (QAP). Program in which the agency or firm responsible for quality assurance maintains detailed monitoring and inspection procedures to ensure conformance with the approved construction documents and referenced standards.

Quality control. Controls and inspections implemented by the fabricator or erector, as applicable, to ensure that the material provided and work performed meet the requirements of the approved construction documents and referenced standards.

Quality control inspector (QCI). Individual designated to perform quality control inspection tasks for the work being performed.

Quality control program (QCP). Program in which the fabricator or erector, as applicable, maintains detailed fabrication or erection and inspection procedures to ensure conformance with the approved design documents, specifications, and referenced standards.

Reentrant. In a cope or weld access hole, a cut at an abrupt change in direction in which the exposed surface is concave.

Registered design professional in responsible charge. A registered design professional engaged by the owner or the owner's authorized agent to review and coordinate certain aspects of the project, as determined by the authority having jurisdiction, for compatibility with the design of the building or structure, including submittal documents prepared by others, deferred submittal documents, and phased submittal documents.

*Required strength**†. Forces, stresses and deformations acting on a structural component, determined by either structural analysis, for the LRFD or ASD load combinations, as applicable, or as specified by this specification or Standard.

Resistance factor, ϕ†. Factor that accounts for unavoidable deviations of the nominal strength from the actual strength and for the manner and consequences of failure.

Restrained construction. Floor and roof assemblies and individual beams in buildings where the surrounding or supporting structure is capable of resisting significant thermal expansion throughout the range of anticipated elevated temperatures.

Reverse curvature. See *double curvature.*

Root of joint. Portion of a joint to be welded where the members are closest to each other.

Rupture strength†. Strength limited by breaking or tearing of members or connecting elements.

Safety factor, Ω†. Factor that accounts for deviations of the actual strength from the nominal strength, deviations of the actual load from the nominal load, uncertainties in the analysis that transforms the load into a load effect, and for the manner and consequences of failure.

Second-order effect. Effect of loads acting on the deformed configuration of a structure; includes P-δ effect and P-Δ effect.

Seismic force-resisting system. That part of the structural system that has been considered in the design to provide the required resistance to the seismic forces prescribed in ASCE/SEI 7.

Seismic response modification factor. Factor that reduces seismic load effects to strength level.

Service load combination. Load combination under which serviceability limit states are evaluated.

Service load†. Load under which serviceability limit states are evaluated.

Serviceability limit state†. Limiting condition affecting the ability of a structure to preserve its appearance, maintainability, durability, comfort of its occupants, or function of machinery, under typical usage.

Shear buckling†. Buckling mode in which a plate element, such as the web of a beam, deforms under pure shear applied in the plane of the plate.

Shear lag. Nonuniform tensile stress distribution in a member or connecting element in the vicinity of a connection.

Shear wall†. Wall that provides resistance to lateral loads in the plane of the wall and provides stability for the structural system.

Shear yielding (punching). In an HSS connection, limit state based on out-of-plane shear strength of the chord wall to which branch members are attached.

Sheet steel. In a composite floor system, steel used for closure plates or miscellaneous trimming in a formed steel deck.

Shim. Thin layer of material used to fill a space between faying or bearing surfaces.

Shop drawings. Drawings of the individual structural steel shipping pieces that are to be produced in the fabrication shop.

Sidesway buckling (frame). Stability limit state involving lateral sidesway instability of a frame.

Simple connection. Connection that transmits negligible bending moment between connected members.

Single-concentrated force. Tensile or compressive force applied normal to the flange of a member.

Single curvature. Deformed shape of a beam with no inflection point within the span.

Slender-element section. Section that is able to only reach a strength limited by local buckling of an element defined by element width to thickness ratio greater than λ_r.

Slip. In a bolted connection, limit state of relative motion of connected parts prior to the attainment of the available strength of the connection.

Slip-critical connection. Bolted connection designed to resist movement by friction on the faying surface of the connection under the clamping force of the bolts.

Slot weld. Weld made in an elongated hole fusing an element to another element.

Snug-tightened joint. Joint with the connected plies in firm contact as specified in Chapter J.

Specifications. The portion of the construction documents that consist of the written requirements for materials, standards and workmanship.

Specified minimum tensile strength. Lower limit of tensile strength specified for a material as defined by ASTM.

Specified minimum yield stress†. Lower limit of yield stress specified for a material as defined by ASTM.

Splice. Connection between two structural elements joined at their ends to form a single, longer element.

Stability. Condition in the loading of a structural component, frame or structure in which a slight disturbance in the loads or geometry does not produce large displacements.

Steel anchor. Headed stud or hot rolled channel welded to a steel member and embedded in the concrete of a composite member to transmit shear, tension or a combination of shear and tension, at the interface of the two materials.

Stiffened element. Flat compression element with adjoining out-of-plane elements along both edges parallel to the direction of loading.

Stiffener. Structural element, typically an angle or plate, attached to a member to distribute load, transfer shear or prevent buckling.

Stiffness. Resistance to deformation of a member or structure, measured by the ratio of the applied force (or moment) to the corresponding displacement (or rotation).

Story drift. Horizontal deflection at the top of the story relative to the bottom of the story.

Story drift ratio. Story drift divided by the story height.

Strain compatibility method. In a composite member, method for determining the stresses considering the stress-strain relationships of each material and its location with respect to the neutral axis of the cross section.

Strength limit state†. Limiting condition in which the maximum strength of a structure or its components is reached.

Stress. Force per unit area caused by axial force, moment, shear or torsion.

Stress concentration. Localized stress considerably higher than average due to abrupt changes in geometry or localized loading.

Strong axis. Major principal centroidal axis of a cross section.

Structural analysis†. Determination of load effects on members and connections based on principles of structural mechanics.

Structural component†. Member, connector, connecting element or assemblage.

Structural Integrity. Performance characteristic of a structure indicating resistance to catastrophic failure.

Structural steel. Steel elements as defined in the AISC *Code of Standard Practice for Steel Buildings and Bridges* Section 2.1.

Structural system. An assemblage of load-carrying components that are joined together to provide interaction or interdependence.

Substantiating connection information. Information submitted by the fabricator in support of connections either selected by the steel detailer or designed by the licensed engineer working for the fabricator.

System imperfection. Initial displacement of points of intersection of members from their nominal locations, such as the out-of-plumbness of columns due to erection tolerances.

T-connection. HSS connection in which the branch member or connecting element is perpendicular to the main member and in which forces transverse to the main member are primarily equilibrated by shear in the main member.

Tensile strength (of material)†. Maximum tensile stress that a material is capable of sustaining as defined by ASTM.

Tensile strength (of member). Maximum tension force that a member is capable of sustaining.

Tension and shear rupture†. In a bolt or other type of mechanical fastener, limit state of rupture due to simultaneous tension and shear force.

Tension field action. Behavior of a panel under shear in which diagonal tensile forces develop in the web and compressive forces develop in the transverse stiffeners in a manner similar to a Pratt truss.

Thermally cut. Cut with gas, plasma or laser.

Tie plate. Plate element used to join two parallel components of a built-up column, girder or strut rigidly connected to the parallel components and designed to transmit shear between them.

Toe of fillet. Junction of a fillet weld face and base metal. Tangent point of a fillet in a rolled shape.

Torsional bracing. Bracing resisting twist of a beam or column.

Torsional buckling†. Buckling mode in which a compression member twists about its shear center axis.

Transverse reinforcement. In an encased composite column, steel reinforcement in the form of closed ties or welded wire fabric providing confinement for the concrete surrounding the steel shape.

Transverse stiffener. Web stiffener oriented perpendicular to the flanges, attached to the web.

Tubing. See *HSS*.

Turn-of-nut method. Procedure whereby the specified pretension in high-strength bolts is controlled by rotating the fastener component a predetermined amount after the bolt has been snug tightened.

Unbraced length. Distance between braced points of a member, measured between the centers of gravity of the bracing members.

Uneven load distribution. In an HSS connection, condition in which the stress is not distributed uniformly through the cross section of connected elements.

Unframed end. The end of a member not restrained against rotation by stiffeners or connection elements.

Unstiffened element. Flat compression element with an adjoining out-of-plane element along one edge parallel to the direction of loading.

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.

Weak axis. Minor principal centroidal axis of a cross section.

Weathering steel. High-strength, low-alloy steel that, with sufficient precautions, is able to be used in typical atmospheric exposures (not marine) without protective paint coating.

Web local crippling†. Limit state of local failure of web plate in the immediate vicinity of a concentrated load or reaction.

Web sidesway buckling. Limit state of lateral buckling of the tension flange opposite the location of a concentrated compression force.

Weld access hole. An opening that permits access for welding, backgouging, or for insertion of backing.

Weld metal. Portion of a fusion weld that has been completely melted during welding. Weld metal has elements of filler metal and base metal melted in the weld thermal cycle.

Weld root. See *root of joint*.

Y-connection. HSS connection in which the branch member or connecting element is not perpendicular to the main member and in which forces transverse to the main member are primarily equilibrated by shear in the main member.

Yield moment†. In a member subjected to bending, the moment at which the extreme outer fiber first attains the yield stress.

Yield point†. First stress in a material at which an increase in strain occurs without an increase in stress as defined by ASTM.

Yield strength†. Stress at which a material exhibits a specified limiting deviation from the proportionality of stress to strain as defined by ASTM.

Yield stress†. Generic term to denote either yield point or yield strength, as applicable for the material.

Yielding†. Limit state of inelastic deformation that occurs when the yield stress is reached.

Yielding (plastic moment)†. Yielding throughout the cross section of a member as the bending moment reaches the plastic moment.

Yielding (yield moment)†. Yielding at the extreme fiber on the cross section of a member when the bending moment reaches the yield moment.

ABBREVIATIONS

The following abbreviations appear in this Specification. The abbreviations are written out where they first appear within a Section.

ACI (American Concrete Institute)
AHJ (authority having jurisdiction)
AISC (American Institute of Steel Construction)
AISI (American Iron and Steel Institute)
ANSI (American National Standards Institute)
ASCE (American Society of Civil Engineers)
ASD (allowable strength design)
ASME (American Society of Mechanical Engineers)
ASNT (American Society for Nondestructive Testing)
AWI (associate welding inspector)
AWS (American Welding Society)
CJP (complete joint penetration)
CVN (Charpy V-notch)
ENA (elastic neutral axis)
EOR (engineer of record)
ERW (electric resistance welded)
FCAW (flux cored arc welding)
FR (fully restrained)
GMAW (gas metal arc welding)
HSLA (high-strength low-alloy)
HSS (hollow structural section)
LRFD (load and resistance factor design)
MT (magnetic particle testing)
NDT (nondestructive testing)
OSHA (Occupational Safety and Health Administration)
PJP (partial joint penetration)
PNA (plastic neutral axis)
PQR (procedure qualification record)
PR (partially restrained)
PT (penetrant testing)
QA (quality assurance)
QAI (quality assurance inspector)
QAP (quality assurance plan)
QC (quality control)
QCI (quality control inspector)
QCP (quality control program)
RCSC (Research Council on Structural Connections)
RT (radiographic testing)
SAW (submerged arc welding)
SEI (Structural Engineering Institute)
SFPE (Society of Fire Protection Engineers)
SMAW (shielded metal arc welding)
SWI (senior welding inspector)
UNC (Unified National Coarse)
UT (ultrasonic testing)
WI (welding inspector)
WPQR (welder performance qualification records)
WPS (welding procedure specification)

CHAPTER A

GENERAL PROVISIONS

This chapter states the scope of this Specification, lists referenced specifications, codes and standards, and provides requirements for materials and structural design documents.

The chapter is organized as follows:

A1. SCOPE

The *Specification for Structural Steel Buildings* (ANSI/AISC 360), hereafter referred to as this Specification, shall apply to the design, fabrication, erection, and quality of the structural steel system or systems with structural steel acting compositely with reinforced concrete, where the steel elements are defined in Section 2.1 of the AISC *Code of Standard Practice for Steel Buildings and Bridges* (ANSI/AISC 303), hereafter referred to as the *Code of Standard Practice*.

This Specification includes the Symbols, the Glossary, Abbreviations, Chapters A through N, and Appendices 1 through 8. The Commentary to this Specification and the User Notes interspersed throughout are not part of this Specification. The phrases "is permitted" and "are permitted" in this document identify provisions that comply with this Specification, but are not mandatory.

User Note: User notes are intended to provide concise and practical guidance in the application of the Specification provisions.

This Specification sets forth criteria for the design, fabrication, and erection of structural steel buildings and other structures, where other structures are defined as structures designed, fabricated, and erected in a manner similar to buildings, with building-like vertical and lateral load-resisting elements.

Wherever this Specification refers to the applicable building code and there is none, the loads, load combinations, system limitations, and general design requirements shall be those in ASCE *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE/SEI 7).

Where conditions are not covered by this Specification, designs are permitted to be based on tests or analysis, subject to the approval of the authority having jurisdiction. Alternative methods of analysis and design are permitted, provided such alternative methods or criteria are acceptable to the authority having jurisdiction.

User Note: For the design of cold-formed steel structural members, the provisions in the AISI *North American Specification for the Design of Cold-Formed Steel Structural Members* (AISI S100) are recommended, except for cold-formed hollow structural sections (HSS), which are designed in accordance with this Specification.

1. Seismic Applications

The AISC *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341) shall apply to the design, fabrication, erection, and quality of seismic force-resisting systems of structural steel or of structural steel acting compositely with reinforced concrete, unless specifically exempted by the applicable building code.

User Note: ASCE/SEI 7 (Table 12.2-1, Item H) specifically exempts structural steel systems in seismic design categories B and C from the requirements in the AISC *Seismic Provisions for Structural Steel Buildings* if they are designed according to this Specification and the seismic loads are computed using a seismic response modification coefficient, *R*, of 3; composite systems are not covered by this exemption. The *Seismic Provisions for Structural Steel Buildings* do not apply in seismic design category A.

2. Nuclear Applications

The design, fabrication, erection, and quality of safety-related nuclear structures shall comply with the provisions of this Specification as modified by the requirements of the AISC *Specification for Safety-Related Steel Structures for Nuclear Facilities* (ANSI/AISC N690).

A2. REFERENCED SPECIFICATIONS, CODES AND STANDARDS

Comment [DC1]: Section A2 is not included in Draft Public Review One.

The following specifications, codes and standards are referenced in this Specification:

(a) American Concrete Institute (ACI)
- ACI 318-14 *Building Code Requirements for Structural Concrete and Commentary*
- ACI 318M-14 *Metric Building Code Requirements for Structural Concrete and Commentary*
- ACI 349-13 *Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary*
- ACI 349M-13 *Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary (Metric)*

(b) American Institute of Steel Construction (AISC)
- ANSI/AISC 303-16 *Code of Standard Practice for Steel Buildings and Bridges*
- ANSI/AISC 341-16 *Seismic Provisions for Structural Steel Buildings*
- ANSI/AISC N690-12 *Specification for Safety-Related Steel Structures for Nuclear Facilities*
- ANSI/AISC N690s1-15 *Specification for Safety-Related Steel Structures for Nuclear Facilities, Supplement No. 1*

(c) American Iron and Steel Institute (AISI)
- AISI 923-xx *Test Standard for Determining the Strength and Stiffness of Shear Connections of Composite Members*
- AISI 924-xx *Test Standard for Determining the Effective Flexural Stiffness of Composite Members*

(d) American Society of Civil Engineers (ASCE)
ASCE/SEI 7-16 *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*
ASCE/SEI/SFPE 29-05 *Standard Calculation Methods for Structural Fire Protection*

(e) American Society of Mechanical Engineers (ASME)
ASME B18.2.6-10 *Fasteners for Use in Structural Applications*
ASME B46.1-09 *Surface Texture, Surface Roughness, Waviness, and Lay*

(f) American Society for Nondestructive Testing (ASNT)
ANSI/ASNT CP-189-2011 *Standard for Qualification and Certification of Nondestructive Testing Personnel*
Recommended Practice No. SNT-TC-1A-2011 *Personnel Qualification and Certification in Nondestructive Testing*

(g) ASTM International (ASTM)
A6/A6M-14 *Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling*
A36/A36M-14 *Standard Specification for Carbon Structural Steel*
A53/A53M-12 *Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless*
A193/A193M-15 *Standard Specification for Alloy-Steel and Stainless Steel Bolting Materials for High Temperature or High Pressure Service and Other Special Purpose Applications*
A194/A194M-15 *Standard Specification for Carbon Steel, Alloy Steel, and Stainless Steel Nuts for Bolts for High Pressure or High Temperature Service, or Both*
A216/A216M-14e1 *Standard Specification for Steel Castings, Carbon, Suitable for Fusion Welding, for High-Temperature Service*
A242/A242M-13 *Standard Specification for High-Strength Low-Alloy Structural Steel*
A283/A283M-13 *Standard Specification for Low and Intermediate Tensile Strength Carbon Steel Plates*
A307-14 *Standard Specification for Carbon Steel Bolts, Studs, and Threaded Rod, 60,000 PSI Tensile Strength*

User Note: ASTM A325/A325M are now included as a Grade within ASTM F3125.

A354-11 *Standard Specification for Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners*
A370-15 *Standard Test Methods and Definitions for Mechanical Testing of Steel Products*
A449-14 *Standard Specification for Hex Cap Screws, Bolts and Studs, Steel, Heat Treated, 120/105/90 ksi Minimum Tensile Strength, General Use*

User Note: ASTM A490/A490M are now included as a Grade within ASTM F3125.

A500/A500M-13 *Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes*
A501/A501M-14 *Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing*

A502-03(2015) *Standard Specification for Rivets, Steel, Structural*

A514/A514M-14 *Standard Specification for High-Yield-Strength, Quenched and Tempered Alloy Steel Plate, Suitable for Welding*

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

A563-15 *Standard Specification for Carbon and Alloy Steel Nuts*

A563M-07(2013) *Standard Specification for Carbon and Alloy Steel Nuts (Metric)*

A568/A568M-15 *Standard Specification for Steel, Sheet, Carbon, Structural, and High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, General Requirements for*

A572/A572M-15 *Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel*

A588/A588M-15 *Standard Specification for High-Strength Low-Alloy Structural Steel, up to 50 ksi [345 MPa] Minimum Yield Point, with Atmospheric Corrosion Resistance*

A606/A606M-15 *Standard Specification for Steel, Sheet and Strip, High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance*

A618/A618M-04(2015) *Standard Specification for Hot-Formed Welded and Seamless High-Strength Low-Alloy Structural Tubing*

A668/A668M-15 *Standard Specification for Steel Forgings, Carbon and Alloy, for General Industrial Use*

A673/A673M-07(2012) *Standard Specification for Sampling Procedure for Impact Testing of Structural Steel*

A709/A709M-13a *Standard Specification for Structural Steel for Bridges*

A751-14a *Standard Test Methods, Practices, and Terminology for Chemical Analysis of Steel Products*

A847/A847M-14 *Standard Specification for Cold-Formed Welded and Seamless High-Strength, Low-Alloy Structural Tubing with Improved Atmospheric Corrosion Resistance*

A913/A913M-15 *Standard Specification for High-Strength Low-Alloy Steel Shapes of Structural Quality, Produced by Quenching and Self-Tempering Process (QST)*

A992/A992M-11(2015) *Standard Specification for Structural Steel Shapes*

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

A1043/A1043M-14 *Standard Specification for Structural Steel with Low Yield to Tensile Ratio for Use in Buildings*

A1065/A1065M-15 *Standard Specification for Cold-Formed Electric-Fusion (Arc) Welded High-Strength Low-Alloy Structural Tubing in Shapes, with 50 ksi [345 MPa] Minimum Yield Point*

A1066/A1066M-11(2015)e1 *Standard Specification for High-Strength Low-Alloy Structural Steel Plate Produced by Thermo-Mechanical Controlled Process (TMCP)*

A1085/A1085M-13 *Standard Specification for Cold-Formed Welded Carbon Steel Hollow Structural Sections (HSS)*

C567/C567M-14 *Standard Test Method for Determining Density of Structural Lightweight Concrete*

E119-15 *Standard Test Methods for Fire Tests of Building Construction and Materials*

E165/E165M-12 *Standard Practice for Liquid Penetrant Examination for General Industry*
E709-15 *Standard Guide for Magnetic Particle Examination*
F436-11 *Standard Specification for Hardened Steel Washers*
F436M-11 *Standard Specification for Hardened Steel Washers (Metric)*
F606/F606M-14a *Standard Test Methods for Determining the Mechanical Properties of Externally and Internally Threaded Fasteners, Washers, Direct Tension Indicators, and Rivets*
F844-07a(2013) *Standard Specification for Washers, Steel, Plain (Flat), Unhardened for General Use*
F959-15 *Standard Specification for Compressible-Washer-Type Direct Tension Indicators for Use with Structural Fasteners*
F959M-13 *Standard Specification for Compressible-Washer-Type Direct Tension Indicators for Use with Structural Fasteners (Metric)*
F1554-15 *Standard Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength*

User Note: ASTM F1554 is the most commonly referenced specification for anchor rods. Grade and weldability must be specified.

User Note: ASTM F1852 and F2280 are now included as Grades within ASTM F3125.

F3043-14e1 *Standard Specification for "Twist Off" Type Tension Control Structural Bolt/Nut/Washer Assemblies, Alloy Steel, Heat Treated, 200 ksi Minimum Tensile Strength*
F3111-14 *Standard Specification for Heavy Hex Structural Bolt/Nut/Washer Assemblies, Alloy Steel, Heat Treated, 200 ksi Minimum Tensile Strength*
F3125/F3125M-15 *Standard Specification for High Strength Structural Bolts, Steel and Alloy Steel, Heat Treated, 120 ksi (830 MPa) and 150 ksi (1040 MPa) Minimum Tensile Strength, Inch and Metric Dimensions*
F3148-17a *Standard Specification for High Strength Structural Bolt Assemblies, Steel and Alloy Steel, Heat Treated, 144 ksi Minimum Tensile Strength, Inch Dimensions*

(h) American Welding Society (AWS)
AWS A5.1/A5.1M:2012 *Specification for Carbon Steel Electrodes for Shielded Metal Arc Welding*
AWS A5.5/A5.5M:2014 *Specification for Low-Alloy Steel Electrodes for Shielded Metal Arc Welding*
AWS A5.17/A5.17M:1997 (R2007) *Specification for Carbon Steel Electrodes and Fluxes for Submerged Arc Welding*
AWS A5.18/A5.18M:2005 *Specification for Carbon Steel Electrodes and Rods for Gas Shielded Arc Welding*
AWS A5.20/A5.20M:2005 (R2015) *Specification for Carbon Steel Electrodes for Flux Cored Arc Welding*
AWS A5.23/A5.23M:2011 *Specification for Low-Alloy Steel Electrodes and Fluxes for Submerged Arc Welding*
AWS A5.25/A5.25M:1997 (R2009) *Specification for Carbon and Low-Alloy Steel Electrodes and Fluxes for Electroslag Welding*
AWS A5.26/A5.26M:1997 (R2009) *Specification for Carbon and Low-Alloy Steel Electrodes for Electrogas Welding*
AWS A5.28/A5.28M:2005 (R2015) *Specification for Low-Alloy Steel Electrodes and Rods for Gas Shielded Arc Welding*

AWS A5.29/A5.29M:2010 *Specification for Low-Alloy Steel Electrodes for Flux Cored Arc Welding*
AWS A5.32/A5.32M:2011 *Welding Consumables—Gases and Gas Mixtures for Fusion Welding and Allied Processes*
AWS A5.36/A5.36M:2012 *Specification for Carbon and Low-Alloy Steel Flux Cored Electrodes for Flux Cored Arc Welding and Metal Cored Electrodes for Gas Metal Arc Welding*
AWS B5.1:2013-AMD1 *Specification for the Qualification of Welding Inspectors*
AWS D1.1/D1.1M:2015 *Structural Welding Code—Steel*
AWS D1.3/D1.3M:2008 *Structural Welding Code—Sheet Steel*

(i) Research Council on Structural Connections (RCSC)
Specification for Structural Joints Using High-Strength Bolts, 2014

(j) Steel Deck Institute (SDI)
ANSI/SDI QA/QC-2011 *Standard for Quality Control and Quality Assurance for Installation of Steel Deck*

A3. MATERIAL

1. Structural Steel Materials

Material test reports or reports of tests made by the fabricator or a testing laboratory shall constitute sufficient evidence of conformity with one of the ASTM standards listed in Section A3.1a. For hot-rolled structural shapes, plates, and bars, such tests shall be made in accordance with ASTM A6/A6M; for sheets, such tests shall be made in accordance with ASTM A568/A568M; for tubing and pipe, such tests shall be made in accordance with the requirements of the applicable ASTM standards listed above for those product forms.

1a. ASTM Designations

Structural steel material conforming to one of the following ASTM specifications is approved for use under this Specification:

(a) Hot-rolled structural shapes
ASTM A36/A36M
ASTM A529/A529M
ASTM A572/A572M
ASTM A588/A588M
ASTM A709/A709M, Grades 36 [250], 50 [345], 50S [345S], 50W [345W], HPS 50W [HPS345W], QST 50 [QST345], QST 50S [QST345S], QST 65 [QST450], QST 70 [QST485]
ASTM A913/A913M
ASTM A992/ A992M
ASTM A1043/A1043M

(b) Hollow structural sections (HSS)
ASTM A53/A53M Grade B
ASTM A500/A500M
ASTM A501/A501M
ASTM A618/A618M
ASTM A847/A847M
ASTM A1065/A1065M

ASTM A1085/A1085M

(c) Plates
ASTM A36/A36M
ASTM A242/A242M
ASTM A283/A283M
ASTM A514/A514M
ASTM A529/A529M
ASTM A572/A572M
ASTM A588/A588M
ASTM A709/A709M
ASTM A1043/A1043M
ASTM A1066/A1066M

(d) Bars
ASTM A36/A36M
ASTM A529/A529M
ASTM A572/A572M
ASTM A709/A709M

(e) Sheets
ASTM A606/A606M
ASTM A1011/A1011M SS, HSLAS, AND HSLAS-F

User Note: Plates, sheets, strips and bars are different products, however, design rules do not make a differentiation between these products.

1b. Unidentified Steel

Unidentified steel, free of injurious defects, is permitted to be used only for members or details whose failure will not reduce the strength of the structure, either locally or overall. Such use shall be subject to the approval of the engineer of record.

User Note: Unidentified steel may be used for details where the precise mechanical properties and weldability are not of concern. These are commonly curb plates, shims and other similar pieces.

1c. Rolled Heavy Shapes

ASTM A6/A6M hot-rolled shapes with a flange thickness exceeding 2 in. (50 mm) are considered to be rolled heavy shapes. Rolled heavy shapes used as members subject to primary (computed) tensile forces due to tension or flexure and spliced or connected using complete-joint-penetration groove welds that fuse through the thickness of the flange or the flange and the web, shall be specified as follows. The structural design documents shall require that such shapes be supplied with Charpy V-notch (CVN) impact test results in accordance with ASTM A6/A6M, Supplementary Requirement S30, Charpy V-Notch Impact Test for Structural Shapes—Alternate Core Location. The impact test shall meet a minimum average value of 20 ft-lb (27 J) absorbed energy at a maximum temperature of +70 °F (+21 °C).

The requirements in this section do not apply if the splices and connections are made by bolting. Where a rolled heavy shape is welded to the surface of another shape using groove welds, the requirements apply only to the shape that has weld metal fused through the cross section.

User Note: Additional requirements for rolled heavy-shape welded joints are given in Sections J1.5, J1.6, J2.6 and M2.2.

1d. Built-Up Heavy Shapes

Built-up cross sections consisting of plates with a thickness exceeding 2 in. (50 mm) are considered built-up heavy shapes. Built-up heavy shapes used as members subject to primary (computed) tensile forces due to tension or flexure and spliced or connected to other members using complete-joint-penetration groove welds that fuse through the thickness of the plates, shall be specified as follows. The structural design documents shall require that the steel be supplied with Charpy V-notch impact test results in accordance with ASTM A6/A6M, Supplementary Requirement S5, Charpy V-Notch Impact Test. The impact test shall be conducted in accordance with ASTM A673/A673M, Frequency P, and shall meet a minimum average value of 20 ft-lb (27 J) absorbed energy at a maximum temperature of +70°F (+21°C).

When a built-up heavy shape is welded to the face of another member using groove welds, these requirements apply only to the shape that has weld metal fused through the cross section.

User Note: Additional requirements for built-up heavy-shape welded joints are given in Sections J1.5, J1.6, J2.6 and M2.2.

2. Steel Castings and Forgings

Steel castings and forgings shall conform to an ASTM standard intended for structural applications and shall provide strength, ductility, weldability and toughness adequate for the purpose. Test reports produced in accordance with the ASTM reference standards shall constitute sufficient evidence of conformity with such standards.

3. Bolts, Washers and Nuts

Bolt, washer and nut material conforming to one of the following ASTM specifications is approved for use under this Specification:

User Note: ASTM F3125 is an umbrella standard that incorporates Grades A325, A325M, A490, A490M, F1852 and F2280, which were previously separate standards.

(a) Bolts

ASTM A307
ASTM A354
ASTM A449
ASTM F3043
ASTM F3111
ASTM F3125/F3125M
ASTM F3148

(b) Nuts
ASTM A194/A194M
ASTM A563
ASTM A563M

(c) Washers
ASTM F436
ASTM F436M
ASTM F844

(d) Compressible-Washer-Type Direct Tension Indicators
ASTM F959
ASTM F959M

Manufacturer's certification shall constitute sufficient evidence of conformity with the standards.

4. Anchor Rods and Threaded Rods

Anchor rod and threaded rod material conforming to one of the following ASTM specifications is approved for use under this Specification:

ASTM A36/A36M
ASTM A193/A193M
ASTM A354
ASTM A449
ASTM A572/A572M
ASTM A588/A588M
ASTM F1554

User Note: ASTM F1554 is the preferred material specification for anchor rods.

ASTM A449 material is permitted for high-strength anchor rods and threaded rods of any diameter.

Threads on anchor rods and threaded rods shall conform to Class 2A, Unified Coarse Thread Series of ASME B1.1 except for anchor rods over 1 in. diameter are permitted to conform to Class 2A, 8UN Thread Series.

Manufacturer's certification shall constitute sufficient evidence of conformity with the standards.

5. Consumables for Welding

Filler metals and fluxes shall conform to one of the following specifications of the American Welding Society:

AWS A5.1/A5.1M
AWS A5.5/A5.5M
AWS A5.17/A5.17M
AWS A5.18/A5.18M
AWS A5.20/A5.20M
AWS A5.23/A5.23M
AWS A5.25/A5.25M
AWS A5.26/A5.26M
AWS A5.28/A5.28M
AWS A5.29/A5.29M
AWS A5.32/A5.32M
AWS A5.36/A5.36M

Manufacturer's certification shall constitute sufficient evidence of conformity with the standards.

6. Headed Stud Anchors

Steel headed stud anchors shall conform to the requirements of the *Structural Welding Code—Steel* (AWS D1.1/D1.1M).

Manufacturer's certification shall constitute sufficient evidence of conformity with AWS D1.1/D1.1M.

A4. STRUCTURAL DESIGN DOCUMENTS AND SPECIFICATIONS

The issuing entity shall obtain authorization from the engineer of record prior to each issuance of the structural design documents and specifications. When authorized, the documents that are released shall be clearly identified with the authorized purpose and shall include the date of release.

Structural design documents and specifications issued for construction of all or a portion of the work shall be clearly legible and drawn to an identified scale that is appropriate to clearly convey the information. They shall be based on the consideration of the design loads, forces, and deformations to be resisted by the structural frame in the completed project and give the following information, as applicable, to define the scope of the work to be fabricated and erected:

(a) Information as required by the applicable building code
(b) Statement of the method of design used: LRFD or ASD
(c) The section, size, material grade, and location of all members
(d) All geometry and work points necessary for layout
(e) Column base, floor, and roof elevation
(f) Column centers and offsets
(g) Identification of the lateral force-resisting system and connecting diaphragm elements that provide for lateral strength and stability in the completed structure
(h) Design provisions for initial imperfections, if different than specified in Chapter C for stability design
(i) Fabrication and erection tolerances not included in or different from the *Code of Standard Practice*
(j) Any special erection conditions or other considerations that are required by the design concept, such as identification of a condition when the structural steel frame in the fully erected and fully connected state requires interaction with nonstructural steel elements for strength or stability, 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
(k) Preset elevation requirements, if any, at free ends of cantilevered members relative to their fixed-end elevations
(l) Column differential shortening information, including performance requirements for monitoring and adjusting for column differential shortening
(m) Requirements for all connections and member reinforcement
(n) Joining requirements between elements of built-up members
(o) Camber requirements for members, including magnitude, direction, and location

(p) Requirements for material grade, size, capacity, and detailing of steel headed stud anchors as specified in Chapter I
(q) Anticipated deflections and the associated loading conditions for major structural elements (such as transfer girders and trusses) that support columns and hangers
(r) Requirements for openings in structural steel members for other trades
(s) Shop painting and surface preparation requirements as required for the design of bolted connections
(t) Requirements for approval documents in addition to what is specified in the *Code of Standard Practice* Section 4.
(u) Charpy V-Notch toughness (CVN) requirements for rolled heavy shapes or built-up heavy shapes, if different than what is required in Section A3
(v) Identification of members and joints subjected to fatigue
(w) Identification of members and joints requiring nondestructive testing in addition to what is required in Chapter N
(x) Additional project requirements, as deemed appropriate by the engineer of record, that impact the life safety of the structure

When structural steel connection design is delegated, the design documents shall include:

(a) Design requirements for the delegated design
(b) Requirements for substantiating connection information

User Note: For projects that require consideration of seismic provisions, additional requirements for information to be shown are contained in Section A4 of the AISC *Seismic Provisions for Structural Steel Buildings*.

A5. APPROVALS

The engineer of record or registered design professional in responsible charge, as applicable, shall require submission of approval documents and shall review and approve, reject, or provide review comments on the approval documents.

When structural steel connection design is delegated to a licensed engineer working with the fabricator, the engineer of record shall require submission of the substantiating connection information and shall review the information submitted for compliance with the information requested. The review shall confirm the following:

(a) The substantiating connection information has been prepared by a licensed engineer
(b) The substantiating connection information conforms to the design documents and specifications
(c) The connection design work conforms to the design intent of the engineer of record on the overall project

User Note: Communication requirements among the parties involved in the approval process are discussed in the AISC *Code of Standard Practice* Section 4. The Commentary to Section 4.1 recommends that a pre-detailing conference be held to facilitate good communication among the parties regarding the engineer's design intent, requests for information (RFI), and the approval documents required for a project.

CHAPTER B

DESIGN REQUIREMENTS

This chapter addresses general requirements for the design of steel structures applicable to all chapters of this Specification.

The chapter is organized as follows:

B1. GENERAL PROVISIONS

The design of members and connections shall be consistent with the intended behavior of the structural system and the assumptions made in the structural analysis.

B2. LOADS AND LOAD COMBINATIONS

The loads, nominal loads, and load combinations shall be those stipulated by the applicable building code. In the absence of a building code, the loads, nominal loads, and load combinations shall be those stipulated in *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE/SEI 7).

User Note: When using ASCE/SEI 7 for design according to Section B3.1 (LRFD), the load combinations in ASCE/SEI 7 Section 2.3 apply. For design, according to Section B3.2 (ASD), the load combinations in ASCE/SEI 7 Section 2.4 apply.

B3. DESIGN BASIS

Design shall be such that no applicable strength or serviceability limit state shall be exceeded when the structure is subjected to all applicable load combinations.

Design for strength shall be performed according to the provisions for load and resistance factor design (LRFD) or to the provisions for allowable strength design (ASD).

User Note: The term "design," as used in this Specification, is defined in the Glossary.

1. Design for Strength Using Load and Resistance Factor Design (LRFD)

Design according to the provisions for load and resistance factor design (LRFD) satisfies the requirements of this Specification when the design strength of each structural component equals or exceeds the required strength determined on the basis of the LRFD load combinations. All provisions of this Specification, except for those in Section B3.2, shall apply.

Design shall be performed in accordance with Equation B3-1:

$$R_u \leq \phi R_n \tag{B3-1}$$

where

R_u = required strength using LRFD load combinations
R_n = nominal strength
ϕ = resistance factor
ϕR_n = design strength

The nominal strength, R_n, and the resistance factor, ϕ, for the applicable limit states are specified in Chapters D through K.

2. Design for Strength Using Allowable Strength Design (ASD)

Design according to the provisions for allowable strength design (ASD) satisfies the requirements of this Specification when the allowable strength of each structural component equals or exceeds the required strength determined on the basis of the ASD load combinations. All provisions of this Specification, except those of Section B3.1, shall apply.

Design shall be performed in accordance with Equation B3-2:

$$R_a \leq \frac{R_n}{\Omega} \tag{B3-2}$$

where

R_a = required strength using ASD load combinations
R_n = nominal strength
Ω = safety factor
R_n/Ω = allowable strength

The nominal strength, R_n, and the safety factor, Ω, for the applicable limit states are specified in Chapters D through K.

3. Required Strength

The required strength of structural members and connections shall be determined by structural analysis for the applicable load combinations, as stipulated in Section B2.

Design by elastic or inelastic analysis is permitted. Requirements for analysis are stipulated in Chapter C and Appendix 1.

4. Design of Connections and Supports

Connection elements shall be designed in accordance with the provisions of Chapters J and K. The forces and deformations used in design of the connections shall be consistent with the intended performance of the connection and the assumptions used in the design of the structure. Self-limiting inelastic deformations of the connections are permitted.

At points of support, beams, girders, and trusses shall be restrained against rotation about their longitudinal axis unless it can be shown by analysis that the restraint is not required.

User Note: *Code of Standard Practice* Section 3.1.2 addresses communication of necessary information for the design of connections.

4a. Simple Connections

A simple connection transmits a negligible moment. In the analysis of the structure, simple connections may be assumed to allow unrestrained relative rotation between the framing elements being connected. A simple connection shall have sufficient rotation capacity to accommodate the required rotation determined by the analysis of the structure.

4b. Moment Connections

Two types of moment connections, fully restrained and partially restrained, are permitted, as specified below.

(a) Fully Restrained (FR) Moment Connections

A fully restrained (FR) moment connection transfers moment with a negligible rotation between the connected members. In the analysis of the structure, the connection may be assumed to allow no relative rotation. An FR connection shall have sufficient strength and stiffness to maintain the initial angle between the connected members at the strength limit states.

(b) Partially Restrained (PR) Moment Connections

Partially restrained (PR) moment connections transfer moments, but the relative rotation between connected members is not negligible. In the analysis of the structure, the moment-rotation response characteristics of any PR connection shall be included. The response characteristics of the PR connection shall be based on the technical literature or established by analytical or experimental means. The component elements of a PR connection shall have sufficient strength, stiffness, and deformation capacity such that the moment-rotation response can be realized up to and including the required strength of the connection.

5. Design of Diaphragms and Collectors

Diaphragms and collectors shall be designed for forces that result from loads, as stipulated in Section B2. They shall be designed in conformance with the provisions of Chapters C through K, as applicable.

6. Design of Anchorages to Concrete

Anchorage between steel and concrete acting compositely shall be designed in accordance with Chapter I. The design of column bases, and anchor rods shall be in accordance with Chapter J.

7. Design for Stability

The structure and its elements shall be designed for stability in accordance with Chapter C.

8. Design for Serviceability

The overall structure and the individual members and connections shall be evaluated for serviceability limit states in accordance with Chapter L.

9. Design for Structural Integrity

When design for structural integrity is required by the applicable building code, the requirements in this section shall be met.

(a) Column splices shall have a nominal tensile strength equal to or greater than $D + L$ for the area tributary to the column between the splice and the splice or base immediately below,
where
D = nominal dead load, kips (N)
L = nominal live load, kips (N)

(b) Beam and girder end connections shall have a minimum nominal axial tensile strength equal to (i) two-thirds of the required vertical shear strength for design according to Section B3.1 (LRFD) or (ii) the required vertical shear strength for design according to Section B3.2 (ASD), but not less than 10 kips in either case.

(c) End connections of members bracing columns shall have a nominal tensile strength equal to or greater than (i) 1% of two-thirds of the required column axial strength at that level for design according to Section B3.1 (LRFD) or (ii) 1% of the required column axial strength at that level for design according to Section B3.2 (ASD).

The strength requirements for structural integrity in this section shall be evaluated independently of other strength requirements. For the purpose of satisfying these requirements, bearing bolts in connections with short-slotted holes parallel to the direction of the tension force and inelastic deformation of the connection are permitted.

10. Design for Ponding

The roof system shall be investigated through structural analysis to ensure stability and strength under ponding conditions unless the roof surface is configured to prevent the accumulation of water.

Ponding stability and strength analysis shall consider the effect of the deflections of the roof's structural framing under all loads (including dead loads) present at the onset of ponding and the subsequent accumulation of rainwater and snowmelt.

The nominal strength and resistance or safety factors for the applicable limit states are specified in Chapters D through K.

11. Design for Fatigue

For members and their connections subjected to repeated loading, fatigue shall be considered in accordance with Appendix 3. Fatigue need not be considered for seismic effects or for the effects of wind loading on typical building lateral force-resisting systems and building enclosure components.

12. Design for Fire Conditions

Two methods of design for fire conditions are provided in Appendix 4: (a) by analysis and (b) by qualification testing. Compliance with the fire-protection requirements in the applicable building code shall be deemed to satisfy the requirements of Appendix 4.

This section is not intended to create or imply a contractual requirement for the engineer of record responsible for the structural design or any other member of the design team.

User Note: Design by qualification testing is the prescriptive method specified in most building codes. Traditionally, on most projects where the architect is the prime professional, the architect has been the responsible party to specify and coordinate fire protection requirements. Design by analysis is a newer engineering approach to fire-protection. Designation of the person(s) responsible for designing for fire conditions is a contractual matter to be addressed on each project.

13. Design for Corrosion Effects

Where corrosion could impair the strength or serviceability of a structure, structural components shall be designed to tolerate corrosion or shall be protected against corrosion.

B4. MEMBER PROPERTIES

1. Classification of Sections for Local Buckling

For members subject to axial compression, sections are classified as nonslender-element or slender-element sections. For a nonslender-element section, the width-to-thickness ratios of its compression elements shall not exceed λ_r from Table B4.1a. If the width-to-thickness ratio of any compression element exceeds λ_r, the section is a slender-element section.

For members subject to flexure, sections are classified as compact, noncompact or slender-element sections. For all sections addressed in Table B4.1b, flanges must be continuously connected to the web or webs. For a section to qualify as compact, the width-to-thickness ratios of its compression elements shall not exceed the limiting width-to-thickness ratios, λ_p, from Table B4.1b. If the width-to-thickness ratio of one or more compression elements exceeds λ_p, but does not exceed λ_r from Table B4.1b, the section is noncompact. If the width-to-thickness ratio of any compression element exceeds λ_r, the section is a slender-element section.

For cases where the web and flange are not continuously attached, consideration of element slenderness must account for the unattached length of the elements and the appropriate plate buckling boundary conditions.

User Note: The Commentary discusses element slenderness when web and flange are not continuously attached.

1a. Unstiffened Elements

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

(a) For flanges of I-shaped members and tees, the width, *b,* is one-half the full-flange width, b_f.

(b) For legs of angles and flanges of channels and zees, the width, *b,* is the full leg or flange width.

(c) For plates, the width, *b*, is the distance from the free edge to the first row of fasteners or line of welds.

(d) For stems of tees, *d* is the full depth of the section.

User Note: Refer to Table B4.1 for the graphic representation of unstiffened element dimensions.

1b. Stiffened Elements

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

(a) For webs of rolled sections, *h* is the clear distance between flanges less the fillet at each flange; h_c is twice the distance from the centroid to the inside face of the compression flange less the fillet or corner radius.

(b) For webs of built-up sections, *h* is the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used, and h_c is twice the distance from the centroid to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used; h_p is twice the distance from the plastic neutral axis to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used.

(c) For flange plates in built-up sections, the width, *b*, is the distance between adjacent lines of fasteners or lines of welds.

(d) For flanges of rectangular hollow structural sections (HSS), the width, *b*, is the clear distance between webs less the inside corner radius on each side. For webs of rectangular HSS, *h* is the clear distance between the flanges less the inside corner radius on each side. If the corner radius is not known, *b* and *h* shall be taken as the corresponding outside dimension minus three times the thickness. The thickness, *t*, shall be taken as the design wall thickness, per Section B4.2.

(e) For flanges or webs of box sections and other stiffened elements, the width, *b*, is the clear distance between the elements providing stiffening.

(f) For perforated cover plates, *b* is the transverse distance between the nearest line of fasteners, and the net area of the plate is taken at the widest hole.

(g) For round hollow structural sections (HSS), the width shall be taken as the outside diameter, *D*, and the thickness, *t*, shall be taken as the design wall thickness, as defined in Section B4.2.

User Note: Refer to Table B4.1 for the graphic representation of stiffened element dimensions.

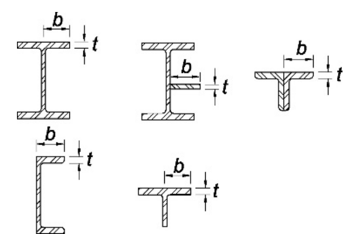
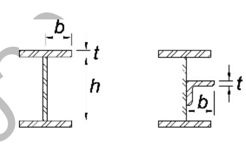
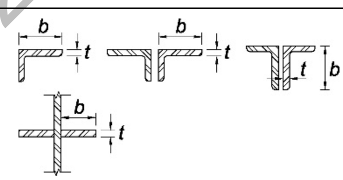
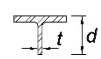
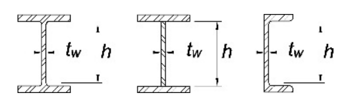
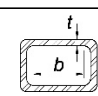
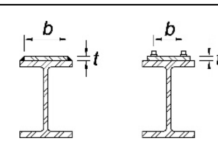
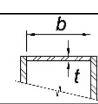
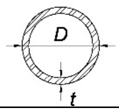
For tapered flanges of rolled sections, the thickness is the nominal value halfway between the free edge and the corresponding face of the web.

2. Design Wall Thickness for HSS

The design wall thickness, *t*, shall be used in calculations involving the wall thickness of hollow structural sections (HSS). The design wall thickness, *t*, shall be taken equal to the nominal thickness for box sections and HSS produced according to ASTM A1065/A1065M or ASTM A1085/A1085M. For HSS produced according to other standards approved for use under this Specification, the design wall thickness, *t*, shall be taken equal to 0.93 times the nominal wall thickness.

User Note: A pipe can be designed using the provisions of this Specification for round HSS sections as long as the pipe conforms to ASTM A53/A53M Grade B and the appropriate limitations of this Specification are used.

TABLE B4.1a
Width-to-Thickness Ratios: Compression Elements Members Subject to Axial Compression

	Case	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio λ_r (nonslender/slender)	Examples
Unstiffened Elements	1	Flanges of rolled I-shaped sections, plates projecting from rolled I-shaped sections, outstanding legs of pairs of angles connected with continuous contact, flanges of channels, and flanges of tees	b/t	$0.56\sqrt{\dfrac{E}{F_y}}$	
	2	Flanges of built-up I-shaped sections and plates or angle legs projecting from built-up I-shaped sections	b/t	[a] $0.64\sqrt{\dfrac{k_c E}{F_y}}$	
	3	Legs of single angles, legs of double angles with separators, and all other unstiffened elements	b/t	$0.45\sqrt{\dfrac{E}{F_y}}$	
	4	Stems of tees	d/t	$0.75\sqrt{\dfrac{E}{F_y}}$	
Stiffened Elements	5	Webs of doubly symmetric rolled and built-up I-shaped sections and channels	h/t_w	$1.49\sqrt{\dfrac{E}{F_y}}$	
	6	Walls of rectangular HSS	b/t	$1.40\sqrt{\dfrac{E}{F_y}}$	
	7	Flange cover plates between lines of fasteners or welds	b/t	$1.40\sqrt{\dfrac{E}{F_y}}$	
	8	All other stiffened elements	b/t	$1.49\sqrt{\dfrac{E}{F_y}}$	
	9	Round HSS	D/t	$0.11\dfrac{E}{F_y}$	

[a] $k_c = 4\big/\sqrt{h/t_w}$, but shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes.

TABLE B4.1b
Width-to-Thickness Ratios: Compression Elements Members Subject to Flexure

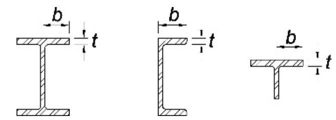
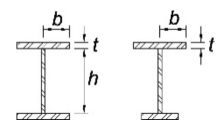
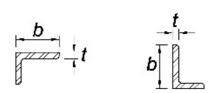
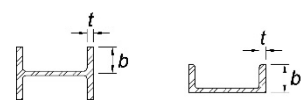
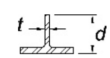
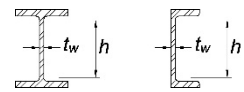
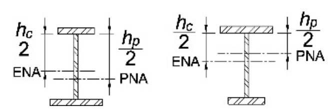
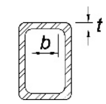
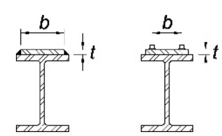
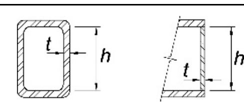
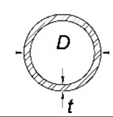
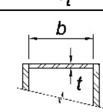
	Case	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio λ_p (compact/noncompact)	Limiting Width-to-Thickness Ratio λ_r (noncompact/slender)	Examples
Unstiffened Elements	10	Flanges of rolled I-shaped sections, channels, and tees	b/t	$0.38\sqrt{\frac{E}{F_y}}$	$1.0\sqrt{\frac{E}{F_y}}$	
Unstiffened Elements	11	Flanges of doubly and singly symmetric I-shaped built-up sections	b/t	$0.38\sqrt{\frac{E}{F_y}}$	[a] [b] $0.95\sqrt{\frac{k_c E}{F_L}}$	
Unstiffened Elements	12	Legs of single angles	b/t	$0.54\sqrt{\frac{E}{F_y}}$	$0.91\sqrt{\frac{E}{F_y}}$	
Unstiffened Elements	13	Flanges of all I-shaped sections and channels in flexure about the minor axis	b/t	$0.38\sqrt{\frac{E}{F_y}}$	$1.0\sqrt{\frac{E}{F_y}}$	
Unstiffened Elements	14	Stems of tees	d/t	$0.84\sqrt{\frac{E}{F_y}}$	$1.52\sqrt{\frac{E}{F_y}}$	

TABLE B4.1b
Width-to-Thickness Ratios: Compression Elements Members Subject to Flexure

	Case	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio λ_p (compact/ noncompact)	Limiting Width-to-Thickness Ratio λ_r (noncompact/ slender)	Examples
Unstiffened Elements	15	Webs of doubly symmetric I-shaped sections and channels	h/t_w	$3.76\sqrt{\frac{E}{F_y}}$	$5.70\sqrt{\frac{E}{F_y}}$	
	16	Webs of singly symmetric I-shaped sections	h_c/t_w	$\dfrac{\frac{h_c}{h_p}\sqrt{\frac{E}{F_y}}}{\left(0.54\frac{M_p}{M_y}-0.09\right)^2} \le \lambda_r$ [c]	$5.70\sqrt{\frac{E}{F_y}}$	
	17	Flanges of rectangular HSS	b/t	$1.12\sqrt{\frac{E}{F_y}}$	$1.40\sqrt{\frac{E}{F_y}}$	
	18	Flange cover plates between lines of fasteners or welds	b/t	$1.12\sqrt{\frac{E}{F_y}}$	$1.40\sqrt{\frac{E}{F_y}}$	
	19	Webs of rectangular HSS and box sections	h/t	$2.42\sqrt{\frac{E}{F_y}}$	$5.70\sqrt{\frac{E}{F_y}}$	
	20	Round HSS	D/t	$0.07\frac{E}{F_y}$	$0.31\frac{E}{F_y}$	
	21	Flanges of box sections	b/t	$1.12\sqrt{\frac{E}{F_y}}$	$1.49\sqrt{\frac{E}{F_y}}$	

[a] $k_c = \dfrac{4}{\sqrt{h/t_w}}$ but shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes.

[b] $F_L = 0.7F_y$ for slender web I-shaped members and major-axis bending of compact and noncompact web built-up I-shaped members with $S_{xt}/S_{xc} \ge 0.7$; and $F_L = F_y S_{xt}/S_{xc} \ge 0.5F_y$ for major-axis bending of compact and noncompact web built-up I-shaped members with $S_{xc}/S_{xt} < 0.7$, where S_{xc}, S_{xt} = elastic section modulus referred to compression and tension flanges, respectively, in.^{3} (mm^3).

[c] M_y is the moment at yielding of the extreme fiber. $M_p = F_y Z_x$, plastic moment, kip-in. (N-mm), where Z_x = plastic section modulus taken about the x-axis, in.^{3} (mm^3)

E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa) ENA = elastic neutral axis

F_y = specified minimum yield stress, ksi (MPa) PNA = plastic neutral axis

3. Gross and Net Area Determination

3a. Gross Area

The gross area, A_g, of a member is the total cross-sectional area.

3b. Net Area

The net area, A_n, of a member is the sum of the products of the thickness and the net width of each element computed as follows:

In computing net area for tension and shear, the width of a bolt hole shall be taken as 1/16 in. (2 mm) greater than the nominal dimension of the hole.

For a chain of holes extending across a part in any diagonal or zigzag line, the net width of the part shall be obtained by deducting from the gross width the sum of the diameters or slot dimensions as provided in this section, of all holes in the chain, and adding, for each gage space in the chain, the quantity $s^2/4g$,

where

g = transverse center-to-center spacing (gage) between fastener gage lines, in. (mm)

s = longitudinal center-to-center spacing (pitch) of any two consecutive holes, in. (mm)

For angles, the gage for holes in opposite adjacent legs shall be the sum of the gages from the back of the angles less the thickness.

For slotted HSS welded to a gusset plate, the net area, A_n, is the gross area minus the product of the thickness and the total width of material that is removed to form the slot.

In determining the net area across plug or slot welds, the weld metal shall not be considered as adding to the net area.

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

B5. FABRICATION AND ERECTION

Fabrication documents, fabrication, shop painting, and erection shall satisfy the requirements stipulated in Chapter M.

User Note: Refer to *Code of Standard Practice* Section 4 addresses requirements for fabrication and erection documents. *Code of Standard Practice* Section 4.4 addresses the approval process for fabrication approval documents.

B6. QUALITY CONTROL AND QUALITY ASSURANCE

Quality control and quality assurance activities shall satisfy the requirements stipulated in Chapter N.

B7. EVALUATION OF EXISTING STRUCTURES

The evaluation of existing structures shall satisfy the requirements stipulated in Appendix 5.

B8. DIMENSIONAL TOLERANCES

The provisions in this Specification are based on the assumption that dimensional tolerances provided in the *Code of Standard Practice* and in the ASTM specifications approved for use under Section A3.1a are satisfied. Where these tolerances are not satisfied, the effect of the out-of-tolerance shall be considered.

CHAPTER C

DESIGN FOR STABILITY

This chapter addresses requirements for the design of structures for stability. The direct analysis method is presented herein.

The chapter is organized as follows:

C1. General Stability Requirements
C2. Calculation of Required Strengths
C3. Calculation of Available Strengths

User Note: Alternative methods for the design of structures for stability are provided in Appendices 1 and 7. Appendix 1 provides alternatives that allow consideration of member imperfections and/or inelasticity directly within the analysis and provides for a more detailed evaluation of the limit states. Appendix 7 provides the effective length method and a first-order elastic method.

C1. GENERAL STABILITY REQUIREMENTS

Stability shall be provided for the structure as a whole and for each of its elements. The effects of all of the following on the stability of the structure and its elements shall be considered: (a) flexural, shear and axial member deformations, and all other component and connection deformations that contribute to the displacements of the structure; (b) second-order effects (including P-Δ and P-δ effects); (c) geometric imperfections; (d) stiffness reductions due to inelasticity, including the effect of partial yielding of the cross section which may be accentuated by the presence of residual stresses; and (e) uncertainty in system, member, and connection strength and stiffness. All load-dependent effects shall be calculated at a level of loading corresponding to LRFD load combinations or 1.6 times ASD load combinations.

Any rational method of design for stability that considers all of the listed effects is permitted; this includes the methods identified in Sections C1.1 and C1.2.

User Note: See Commentary Section C1 and Table C-C1.1 for an explanation of how requirements (a) through (e) of Section C1 are satisfied in the methods of design listed in Sections C1.1 and C1.2.

1. Direct Analysis Method of Design

The direct analysis method of design is permitted for all structures, and can be based on either elastic or inelastic analysis. For design by elastic analysis, required strengths shall be calculated in accordance with Section C2 and the calculation of available strengths in accordance with Section C3. For design by advanced analysis, the provisions of Section 1.1 and Sections 1.2 or 1.3 of Appendix 1 shall be satisfied.

2. Alternative Methods of Design

The effective length method and the first-order analysis method, both defined in Appendix 7, are based on elastic analysis and are permitted as alternatives to the direct analysis method for structures that satisfy the limitations specified in that appendix.

C2. CALCULATION OF REQUIRED STRENGTHS

For the direct analysis method of design, the required strengths of components of the structure shall be determined from an elastic analysis conforming to Section C2.1. The analysis shall include consideration of initial imperfections in accordance with Section C2.2 and adjustments to stiffness in accordance with Section C2.3.

1. General Analysis Requirements

The analysis of the structure shall conform to the following requirements:

(a) The analysis shall consider flexural, shear, and axial member deformations, and all other component and connection deformations that contribute to displacements of the structure. The analysis shall incorporate reductions in all stiffnesses that are considered to contribute to the stability of the structure, as specified in Section C2.3.

(b) The analysis shall be a second-order analysis that considers both P-Δ and P-δ effects, except that it is permissible to neglect the effect of P-δ on the response of the structure when the following conditions are satisfied: (1) the structure supports gravity loads primarily through nominally vertical columns, walls or frames; (2) the ratio of maximum second-order drift to maximum first-order drift (both determined for LRFD load combinations or 1.6 times ASD load combinations, with stiffnesses adjusted as specified in Section C2.3) in all stories is equal to or less than 1.7; and (3) no more than one-third of the total gravity load on the structure is supported by columns that are part of moment-resisting frames in the direction of translation being considered. It is necessary in all cases to consider P-δ effects in the evaluation of individual members subject to compression and flexure.

User Note: A P-Δ-only second-order analysis (one that neglects the effects of P-δ on the response of the structure) is permitted under the conditions listed. In this case, the requirement for considering P-δ effects in the evaluation of individual members can be satisfied by applying the B_1 multiplier defined in Appendix 8, Section 8.1.2, to the required flexural strength of the member.

Use of the approximate method of second-order analysis provided in Appendix 8, Section 8.1, is permitted.

(c) The analysis shall consider all gravity and other applied loads that may influence the stability of the structure.

User Note: It is important to include in the analysis all gravity loads, including loads on leaning columns and other elements that are not part of the lateral force-resisting system.

(d) For design by LRFD, the second-order analysis shall be carried out under LRFD load combinations. For design by 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 of components.

2. Consideration of Initial System Imperfections

The effect of initial imperfections in the position of points of intersection of members on the stability of the structure shall be taken into account either by direct modeling of these imperfections in the analysis as specified in Section C2.2a or by the application of notional loads as specified in Section C2.2b.

> **User Note:** The imperfections required to be considered in this section are imperfections in the locations of points of intersection of members (system imperfections). In typical building structures, the important imperfection of this type is the out-of-plumbness of columns. Consideration of initial out-of-straightness of individual members (member imperfections) is not required in the structural analysis when using the provisions of this section; it is accounted for in the compression member design provisions of Chapter E and need not be considered explicitly in the analysis as long as it is within the limits specified in the *Code of Standard Practice.* Appendix 1, Section 1.2, provides an extension to the direct analysis method that includes modeling of member imperfections (initial out-of-straightness) within the structural analysis.

2a. Direct Modeling of Imperfections

In all cases, it is permissible to account for the effect of initial system imperfections by including the imperfections directly in the analysis. The structure shall be analyzed with points of intersection of members displaced from their nominal locations. The magnitude of the initial displacements shall be the maximum amount considered in the design; the pattern of initial displacements shall be such that it provides the greatest destabilizing effect.

> **User Note:** Initial displacements similar in configuration to both displacements due to loading and anticipated buckling modes should be considered in the modeling of imperfections. The magnitude of the initial displacements should be based on permissible construction tolerances, as specified in the *Code of Standard Practice* or other governing requirements, or on actual imperfections if known.

In the analysis of structures that support gravity loads primarily through nominally vertical columns, walls or frames, where the ratio of maximum second-order story drift to maximum first-order story drift (both determined for LRFD load combinations or 1.6 times ASD load combinations, with stiffnesses adjusted as specified in Section C2.3) in all stories is equal to or less than 1.7, it is permissible to include initial system imperfections in the analysis for gravity-only load combinations and not in the analysis for load combinations that include applied lateral loads.

2b. Use of Notional Loads to Represent Imperfections

For structures that support gravity loads primarily through nominally vertical columns, walls, or frames, it is permissible to use notional loads to represent the effects of initial system imperfections in the position of points of intersection of members in accordance with the requirements of this section. The notional load shall be applied to a model of the structure based on its nominal geometry.

User Note: In general, the notional load concept is applicable to all types of structures and to imperfections in the positions of both points of intersection of members and points along members, but the specific requirements in Sections C2.2b(a) through C2.2b(d) are applicable only for the particular class of structure and type of system imperfection identified here.

(a) Notional loads shall be applied as lateral loads at all levels. The notional loads shall be additive to other lateral loads and shall be applied in all load combinations, except as indicated in Section C2.2b(d). The magnitude of the notional loads shall be:

$$N_i = 0.002\alpha Y_i \tag{C2-1}$$

where

α = 1.0 (LRFD); α = 1.6 (ASD)

N_i = notional load applied at level i, kips (N)

Y_i = gravity load applied at level i from the LRFD load combination or ASD load combination, as applicable, kips (N)

User Note: The use of notional loads can lead to additional (generally small) fictitious base shears in the structure. The correct horizontal reactions at the foundation may be obtained by applying an additional horizontal force at the base of the structure, equal and opposite in direction to the sum of all notional loads, distributed among vertical load-carrying elements in the same proportion as the gravity load supported by those elements. The notional loads can also lead to additional overturning effects, which are not fictitious.

(b) The notional load at any level, N_i, shall be distributed over that level in the same manner as the gravity load at the level. The notional loads shall be applied in the direction that provides the greatest destabilizing effect.

User Note: For most building structures, the requirement regarding notional load direction may be satisfied as follows: for load combinations that do not include lateral loading, consider two alternative orthogonal directions of notional load application, in a positive and a negative sense in each of the two directions, in the same direction at all levels; for load combinations that include lateral loading, apply all notional loads in the direction of the resultant of all lateral loads in the combination.

(c) The notional load coefficient of 0.002 in Equation C2-1 is based on a nominal initial story out-of-plumbness ratio of 1/500; where the use of a different maximum out-of-plumbness is justified, it is permissible to adjust the notional load coefficient proportionally.

User Note: An out-of-plumbness of 1/500 represents the maximum tolerance on column plumbness specified in the *Code of Standard Practice*. In some cases, other specified tolerances, such as those on plan location of columns, will govern and will require a tighter plumbness tolerance.

(d) For structures in which the ratio of maximum second-order drift to maximum first-order drift (both determined for LRFD load combinations or 1.6 times ASD load combinations, with stiffnesses adjusted as specified in Section C2.3) in all stories is equal to or less than 1.7, it is permissible to apply the notional load, N_i, only in gravity-only load combinations and not in combinations that include other lateral loads.

3. Adjustments to Stiffness

The analysis of the structure to determine the required strengths of components shall use reduced stiffnesses, as follows:

(a) A factor of 0.80 shall be applied to all stiffnesses that are considered to contribute to the stability of the structure. It is permissible to apply this reduction factor to all stiffnesses in the structure.

> **User Note:** Applying the stiffness reduction to some members and not others can, in some cases, result in artificial distortion of the structure under load and possible unintended redistribution of forces. This can be avoided by applying the reduction to all members, including those that do not contribute to the stability of the structure.

(b) An additional factor, τ_b, shall be applied to the flexural stiffnesses of all members whose flexural stiffnesses are considered to contribute to the stability of the structure. For noncomposite members, τ_b shall be defined as follows (see Section I1.5 for the definition of τ_b for composite members):

(1) When $\alpha P_r/P_{ns} \le 0.5$

$$\tau_b = 1.0 \qquad \text{(C2-2a)}$$

(2) When $\alpha P_r/P_{ns} > 0.5$

$$\tau_b = 4(\alpha P_r/P_{ns})\left[1-(\alpha P_r/P_{ns})\right] \qquad \text{(C2-2b)}$$

where

α = 1.0 (LRFD); α = 1.6 (ASD)

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

P_{ns} = cross-section compressive strength; for nonslender-element sections, $P_{ns} = F_yA_g$, and for slender-element sections, $P_{ns} = F_yA_e$, where A_e is as defined in Section E7 with $F_{cr} = F_y$, kips (N)

> **User Note**: Taken together, Sections (a) and (b) require the use of 0.8 τ_b times the nominal elastic flexural stiffness and 0.8 times other nominal elastic stiffnesses for structural steel members in the analysis.

(c) In structures to which Section C2.2b is applicable, in lieu of using τ_b < 1.0, where $\alpha P_r/P_{ns} > 0.5$, it is permissible to use τ_b = 1.0 for all noncomposite members if a notional load of $0.001\alpha Y_i$ [where Y_i is as defined in Section C2.2b(a)] is applied at all levels, in the direction specified in Section C2.2b(b), in all load combinations. These notional loads shall be added to those, if any, used to account for the effects of initial imperfections in the position of points of intersection of members and shall not be subject to the provisions of Section C2.2b(d).

(d) Where components comprised of materials other than structural steel are considered to contribute to the stability of the structure, and the governing codes and specifications for the other materials require greater reductions in stiffness, such greater stiffness reductions shall be applied to those components.

C3. CALCULATION OF AVAILABLE STRENGTHS

For the direct analysis method of design, the available strengths of members and connections shall be calculated in accordance with the provisions of Chapters D through K, as applicable, with no further consideration of overall structure stability. The effective length for flexural buckling of all members shall be taken as the unbraced length unless a smaller value is justified by rational analysis.

Bracing intended to define the unbraced lengths of members shall have sufficient stiffness and strength to control member movement at the braced points.

User Note: Methods of satisfying this bracing requirement are provided in Appendix 6. The requirements of Appendix 6 are not applicable to bracing that is included in the design of the lateral force-resisting system of the overall structure.

CHAPTER D

DESIGN OF MEMBERS FOR TENSION

This chapter applies to members subject to axial tension.

The chapter is organized as follows:

User Note: For cases not included in this chapter, the following sections apply:
- B3.11 Members subject to fatigue
- Chapter H Members subject to combined axial tension and flexure
- J3 Threaded rods
- J4.1 Connecting elements in tension
- J4.3 Block shear rupture strength at end connections of tension members

D1. SLENDERNESS LIMITATIONS

There is no maximum slenderness limit for members in tension.

User Note: For members designed on the basis of tension, the slenderness ratio, L/r, preferably should not exceed 300. This suggestion does not apply to rods or hangers in tension.

D2. TENSILE STRENGTH

The design tensile strength, $\phi_t P_n$, and the allowable tensile strength, P_n/Ω_t, of tension members shall be the lower value obtained according to the limit states of tensile yielding in the gross section and tensile rupture in the net section.

(a) For tensile yielding in the gross section

$$P_n = F_y A_g \tag{D2-1}$$

$\phi_t = 0.90$ (LRFD) $\Omega_t = 1.67$ (ASD)

(b) For tensile rupture in the net section

$$P_n = F_u A_e \tag{D2-2}$$

$\phi_t = 0.75$ (LRFD) $\Omega_t = 2.00$ (ASD)

where
A_e = effective net area, in.^{2} (mm^2)
A_g = gross area of member, in.^{2} (mm^2)
F_y = specified minimum yield stress, ksi (MPa)
F_u = specified minimum tensile strength, ksi (MPa)

Where connections use plug, slot or fillet welds in holes or slots, the effective net area through the holes shall be used in Equation D2-2.

D3. EFFECTIVE NET AREA

The gross area, A_g, and net area, A_n, of tension members shall be determined in accordance with the provisions of Section B4.3.

The effective net area of tension members shall be determined as

$$A_e = A_n U \tag{D3-1}$$

where U, the shear lag factor, is determined as shown in Table D3.1.

For open cross sections such as W, M, S, C, or HP shapes, WTs, STs, and single and double angles, the shear lag factor, U, need not be less than the ratio of the gross area of the connected element(s) to the member gross area. This provision does not apply to closed sections, such as HSS sections, nor to plates.

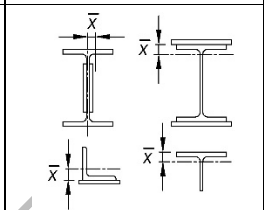
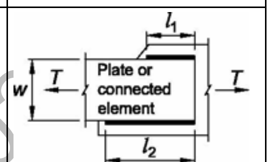
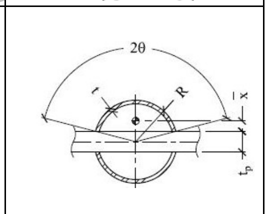
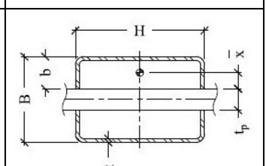
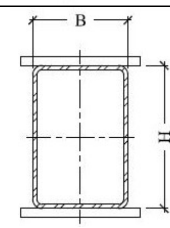
D4. BUILT-UP MEMBERS

For limitations on the longitudinal spacing of connectors between elements in continuous contact consisting of a plate and a shape, or two plates, see Section J3.5.

Lacing, perforated cover plates, or tie plates without lacing are permitted to be used on the open sides of built-up tension members. Tie plates shall have a length not less than two-thirds the distance between the lines of welds or fasteners connecting them to the components of the member. The thickness of such tie plates shall not be less than one-fiftieth of the distance between these lines. The longitudinal spacing of intermittent welds or fasteners at tie plates shall not exceed 6 in. (150 mm).

User Note: The longitudinal spacing of connectors between components should preferably limit the slenderness ratio in any component between the connectors to 300.

TABLE D3.1
Shear Lag Factors for Connections to Tension Members

Case	Description of Element	Shear Lag Factor, U	Examples
1	All tension members where the tension load is transmitted directly to each of the cross-sectional elements by fasteners or welds (except as in Cases 4, 5 and 6).	$U = 1.0$	–
2	All tension members, except HSS, where the tension load is transmitted to some but not all of the cross-sectional elements by fasteners or by longitudinal welds in combination with transverse welds. Alternatively, Case 7 is permitted for W, M, S and HP shapes and Case 8 is permitted for angles.	$U = 1 - \dfrac{\bar{x}}{l}$	
3	All tension members where the tension load is transmitted only by transverse welds to some but not all of the cross-sectional elements.	$U = 1.0$ and A_n = area of the directly connected elements	–
4[a]	Plates, angles, channels with welds at heels, tees, and W-shapes with connected elements, where the tension load is transmitted by longitudinal welds only. See Case 2 for definition of $\bar{x}$.	$U = \dfrac{3l^2}{3l^2 + w^2}\left(1 - \dfrac{\bar{x}}{l}\right)$	
5	Round and rectangular HSS with single concentric gusset through slots in the HSS.	$\bar{x} = \dfrac{R\sin\theta}{\theta} - \dfrac{1}{2}t_p$ θ in rad $U = \left[1 + \left(\dfrac{\bar{x}}{l}\right)^{3.2}\right]^{-10}$	
		$\bar{x} = b - \dfrac{2b^2 + tH - 2t^2}{2H + 4b - 4t}$ $U = 1 - \dfrac{\bar{x}}{l}$	
6	Rectangular HSS with two side gusset plates.	$U = \dfrac{BU_B + HU_H}{H + B}$ $U_B = \dfrac{3l^2}{3l^2 + B^2}$ $U_H = \dfrac{3l^2}{3l^2 + H^2}$	
7	W-, M-, S- or HP-shapes, or tees cut from these shapes. (If U is calculated per Case 2, the larger value is permitted to be used.) — with flange connected with three or more fasteners per line in the direction of loading	$b_f \geq \dfrac{2}{3}d$, $U = 0.90$ $b_f < \dfrac{2}{3}d$, $U = 0.85$	–
	— with web connected with four or more fasteners per line in the direction of loading	$U = 0.70$	–

8	Single and double angles (If U is calculated per Case 2, the larger value is permitted to be used.)	with four or more fasteners per line in the direction of loading	$U = 0.80$	–
		with three fasteners per line in the direction of loading (with fewer than three fasteners per line in the direction of loading, use Case 2)	$U = 0.60$	–

B = overall width of rectangular HSS member, measured 90° to the plane of the connection, in. (mm); D = outside diameter of round HSS, in. (mm); H = overall height of rectangular HSS member, measured in the plane of the connection, in. (mm); d = depth of section, in. (mm); for tees, d = depth of the section from which the tee was cut, in. (mm); l = length of connection, in. (mm); w = width of plate, in. (mm); $\bar{x}$ = eccentricity of connection, in. (mm).

[a] $l = \dfrac{l_1 + l_2}{2}$, where l_1 and l_2 shall not be less than 4 times the weld size.

D5. PIN-CONNECTED MEMBERS

1. Tensile Strength

The design tensile strength, $\phi_t P_n$, and the allowable tensile strength, P_n/Ω_t, of pin-connected members, shall be the lower value determined according to the limit states of tensile rupture, shear rupture, bearing and yielding.

(a) For tensile rupture on the net effective area

$$P_n = F_u(2tb_e) \tag{D5-1}$$

$\phi_t = 0.75$ (LRFD) $\qquad \Omega_t = 2.00$ (ASD)

(b) For shear rupture on the effective area

$$P_n = 0.6F_u A_{sf} \tag{D5-2}$$

$\phi_{sf} = 0.75$ (LRFD) $\qquad \Omega_{sf} = 2.00$ (ASD)

where

$A_{sf} = 2t(a + d/2)$
= area on the shear failure path, in.^{2} (mm^2)
C_r = reduction factor for shear rupture on pin-connected members
= 1.0 when $d_h - d \le 1/32$ in. (1 mm)
= 0.95 when 1/32 in. $< d_h - d \le 1/16$ in. (1 mm $< d_h - d \le 2$ mm)
a = shortest distance from edge of the pin hole to the edge of the member measured parallel to the direction of the force, in. (mm)
b_e = $2t + 0.63$, in. (= $2t + 16$, mm), but not more than the actual distance from the edge of the hole to the edge of the part measured in the direction normal to the applied force, in. (mm)
d = diameter of pin, in. (mm)
d_h = diameter of hole, in. (mm)
t = thickness of plate, in. (mm)

(c) For bearing on the projected area of the pin, use Section J7.
(d) For yielding on the gross section, use Section D2(a).

2. Dimensional Requirements

Pin-connected members shall meet the following requirements:

(a) The pin hole shall be located midway between the edges of the member in the direction normal to the applied force.
(b) When the pin is expected to provide for relative movement between connected parts while under full load, the diameter of the pin hole shall not be more than 1/32 in. (1 mm) greater than the diameter of the pin for pins less than 3 in. in diameter and not more than 1/16 in. (2 mm) greater than the diameter of the pin for pins of 3 in. (75 mm) in diameter or greater.
(c) The width of the plate at the pin hole shall not be less than $2b_e + d$ and the minimum extension, a, beyond the bearing end of the pin hole, parallel to the axis of the member, shall not be less than $1.33b_e$.
(d) The corners beyond the pin hole are permitted to be cut at 45° to the axis of the member, provided the net area beyond the pin hole, on a plane perpendicular to the cut, is not less than that required beyond the pin hole parallel to the axis of the member.

D6. EYEBARS

1. Tensile Strength

The available tensile strength of eyebars shall be determined in accordance with Section D2, with A_g taken as the gross area of the eyebar body.

For calculation purposes, the width of the body of the eyebars shall not exceed eight times its thickness.

2. Dimensional Requirements

Eyebars shall meet the following requirements:

(a) Eyebars shall be of uniform thickness, without reinforcement at the pin holes, and have circular heads with the periphery concentric with the pin hole.

(b) The radius of transition between the circular head and the eyebar body shall not be less than the head diameter.

(c) The pin diameter shall not be less than seven-eighths times the eyebar body width, and the pin-hole diameter shall not be more than 1/32 in. (1 mm) greater than the pin diameter.

(d) For steels having F_y greater than 70 ksi (485 MPa), the hole diameter shall not exceed five times the plate thickness, and the width of the eyebar body shall be reduced accordingly.

(e) A thickness of less than 1/2 in. (13 mm) is permissible only if external nuts are provided to tighten pin plates and filler plates into snug contact.

(f) The width from the hole edge to the plate edge perpendicular to the direction of applied load shall be greater than two-thirds and, for the purpose of calculation, not more than three-fourths times the eyebar body width.

CHAPTER E

DESIGN OF MEMBERS FOR COMPRESSION

This chapter addresses members subject to axial compression.

The chapter is organized as follows:

User Note: For cases not included in this chapter, the following sections apply:
- H1 – H2 Members subject to combined axial compression and flexure
- H3 Members subject to axial compression and torsion
- I2 Composite axially loaded members
- J4.4 Compressive strength of connecting elements

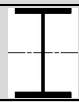
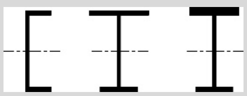
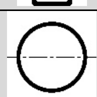
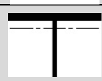
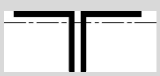
E1. GENERAL PROVISIONS

The design compressive strength, $\phi_c P_n$, and the allowable compressive strength, P_n / Ω_c, are determined as follows.

The nominal compressive strength, P_n, shall be the lowest value obtained based on the applicable limit states of flexural buckling, torsional buckling, and flexural-torsional buckling.

$$\phi_c = 0.90 \text{ (LRFD)} \qquad \Omega_c = 1.67 \text{ (ASD)}$$

TABLE USER NOTE E1.1
Selection Table for the Application of Chapter E Sections

Cross Section	Without Slender Elements		With Slender Elements	
	Sections in Chapter E	Limit States	Sections in Chapter E	Limit States
	E3 E4	FB TB	E7	LB FB TB
	E3 E4	FB FTB	E7	LB FB FTB
	E3	FB	E7	LB FB
	E3	FB	E7	LB FB
	E3 E4	FB FTB	E7	LB FB FTB
	E6 E3 E4	FB FTB	E6 E7	LB FB FTB
	E5		E5	
	E3	FB	N/A	N/A
Unsymmetrical shapes other than single angles	E4	FTB	E7	LB FTB

FB = flexural buckling, TB = torsional buckling, FTB = flexural-torsional buckling, LB = local buckling, N/A = not applicable

E2. EFFECTIVE LENGTH

The effective length, L_c, for calculation of member slenderness, L_c/r, shall be determined in accordance with Chapter C or Appendix 7,

where

L_c = KL = effective length of member, in. (mm)
K = effective length factor
L = laterally unbraced length of the member, in. (mm)
r = radius of gyration, in. (mm)

User Note: For members designed on the basis of compression, the effective slenderness ratio, L_c/r, preferably should not exceed 200.

User Note: The effective length, L_c, may be determined using an effective length factor, K, or a buckling analysis.

E3. FLEXURAL BUCKLING OF MEMBERS WITHOUT SLENDER ELEMENTS

This section applies to nonslender-element compression members, as defined in Section B4.1, for elements in axial compression.

User Note: When the torsional effective length is larger than the lateral effective length, Section E4 may control.

The nominal compressive strength, P_n, shall be determined based on the limit state of flexural buckling:

$$P_n = F_n A_g \tag{E3-1}$$

The nominal stress, F_n, is determined as follows:

(a) When $\dfrac{L_c}{r} \le 4.71\sqrt{\dfrac{E}{F_y}}$ (or $\dfrac{F_y}{F_e} \le 2.25$)

$$F_n = \left(0.658^{\frac{F_y}{F_e}}\right)F_y \tag{E3-2}$$

(b) When $\dfrac{L_c}{r} > 4.71\sqrt{\dfrac{E}{F_y}}$ (or $\dfrac{F_y}{F_e} > 2.25$)

$$F_n = 0.877F_e \tag{E3-3}$$

where

A_g = gross area of member, in.^{2} (mm^2)
E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)
F_e = elastic buckling stress determined according to Equation E3-4; or as specified in Appendix 7, Section 7.2.3(b); or through an elastic buckling analysis, as applicable, ksi (MPa)

$$= \frac{\pi^2 E}{\left(\frac{L_c}{r}\right)^2} \tag{E3-4}$$

F_y = specified minimum yield stress of the type of steel being used, ksi (MPa)
r = radius of gyration, in. (mm)

User Note: The two inequalities for calculating the limits of applicability of Sections E3(a) and E3(b), one based on L_c/r and one based on F_y/F_e, provide the same result for flexural buckling.

E4. TORSIONAL AND FLEXURAL-TORSIONAL BUCKLING OF SINGLE ANGLES AND MEMBERS WITHOUT SLENDER ELEMENTS

This section applies to singly symmetric and unsymmetric members, certain doubly symmetric members, such as cruciform or built-up members, and doubly symmetric members when the torsional unbraced length exceeds the lateral unbraced length, all without slender elements. These provisions also apply to single angles with $b/t > 0.71\sqrt{E/F_y}$, where b is the width of the longest leg and t is the thickness.

The nominal compressive strength, P_n, shall be determined based on the limit states of torsional and flexural-torsional buckling:

$$P_n = F_n A_g \tag{E4-1}$$

The nominal stress, F_n, shall be determined according to Equation E3-2 or E3-3, using the torsional or flexural-torsional elastic buckling stress, F_e, determined as follows:

(a) For doubly symmetric members twisting about the shear center

$$F_e = \left(\frac{\pi^2 E C_w}{L_{cz}^2} + GJ\right)\frac{1}{I_x + I_y} \tag{E4-2}$$

(b) For singly symmetric members twisting about the shear center where y is the axis of symmetry

$$F_e = \left(\frac{F_{ey} + F_{ez}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{ey}F_{ez}H}{(F_{ey} + F_{ez})^2}}\right] \tag{E4-3}$$

User Note: For singly symmetric members with the x-axis as the axis of symmetry, such as channels, Equation E4-3 is applicable with F_{ey} replaced by F_{ex}.

(c) For unsymmetric members twisting about the shear center, F_e is the lowest root of the cubic equation

$$(F_e - F_{ex})(F_e - F_{ey})(F_e - F_{ez}) - F_e^2(F_e - F_{ey})\left(\frac{x_o}{\bar{r}_o}\right)^2 - F_e^2(F_e - F_{ex})\left(\frac{y_o}{\bar{r}_o}\right)^2 = 0 \tag{E4-4}$$

where

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

$$F_{ex} = \frac{\pi^2 E}{\left(\dfrac{L_{cx}}{r_x}\right)^2} \tag{E4-5}$$

$$F_{ey} = \frac{\pi^2 E}{\left(\dfrac{L_{cy}}{r_y}\right)^2} \tag{E4-6}$$

$$F_{ez} = \left(\frac{\pi^2 E C_w}{L_{cz}{}^2} + GJ\right)\frac{1}{A_g \bar{r}_o^2} \tag{E4-7}$$

G = shear modulus of elasticity of steel = 11,200 ksi (77 200 MPa)

H = flexural constant

$$= 1 - \frac{x_o^2 + y_o^2}{\bar{r}_o^2} \tag{E4-8}$$

I_x, I_y = moment of inertia about the principal axes, in.^{4} (mm^4)

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

K_x = effective length factor for flexural buckling about *x*-axis

K_y = effective length factor for flexural buckling about *y*-axis

K_z = effective length factor for torsional buckling about the longitudinal axis

L_{cx} = K_xL_x = effective length of member for buckling about *x*-axis, in. (mm)

L_{cy} = K_yL_y = effective length of member for buckling about *y*-axis, in. (mm)

L_{cz} = K_zL_z = effective length of member for buckling about longitudinal axis, in. (mm)

L_x, L_y, L_z = laterally unbraced length of the member for each axis, in. (mm)

$\bar{r}_o$ = polar radius of gyration about the shear center, in. (mm)

$$\bar{r}_o^2 = x_o^2 + y_o^2 + \frac{I_x + I_y}{A_g} \tag{E4-9}$$

r_x = radius of gyration about *x*-axis, in. (mm)

r_y = radius of gyration about *y*-axis, in. (mm)

x_o, y_o = coordinates of the shear center with respect to the centroid, in. (mm)

User Note: For doubly symmetric I-shaped sections, C_w may be taken as $I_y h_o{}^2/4$, where h_o is the distance between flange centroids, in lieu of a more precise analysis. For tees and double angles, the term with C_w may be omitted when computing F_{ez}.

(d) For doubly symmetric I-shaped members with minor axis lateral bracing offset from the shear center

$$F_e = \left[\frac{\pi^2 E I_y}{L_{cz}^2}\left(\frac{h_o^2}{4} + y_a^2\right) + GJ\right]\frac{1}{A_g r_o^2} \tag{E4-10}$$

where

$$r_o{}^2 = \left(r_x^2 + r_y^2 + y_a^2 + x_a^2\right) \tag{E4-11}$$

h_o = distance between flange centroids, in. (mm)
y_a = bracing offset distance along y-axis, in. (mm)
x_a = bracing offset distance along x-axis = 0

(e) For doubly symmetric I-shaped members with major axis lateral bracing offset from the shear center

$$F_e = \left[\frac{\pi^2 EI_y}{L_{cz}^2}\left(\frac{h_o^2}{4} + \frac{I_x}{I_y}x_a^2\right) + GJ\right]\frac{1}{A_g r_o^2} \quad \text{(E4-12)}$$

where
y_a = bracing offset distance along y-axis = 0
x_a = bracing offset distance along x-axis, in. (mm)

(f) For all other members with lateral bracing offset from the shear center, the elastic buckling stress, F_e, shall be determined by analysis.

User Note: Bracing offset from the shear center is often referred to as constrained-axis torsional buckling and is discussed further in the Commentary. Members that buckle in this mode will exhibit twisting because the braces restrain only lateral movement.

E5. SINGLE-ANGLE COMPRESSION MEMBERS

The nominal compressive strength, P_n, of single-angle members shall be the lowest value based on the limit states of flexural buckling in accordance with Section E3 or Section E7, as applicable, or flexural-torsional buckling in accordance with Section E4. Flexural-torsional buckling need not be considered when $b/t \le 0.71\sqrt{E/F_y}$.

The effects of eccentricity on single-angle members are permitted to be neglected and the member evaluated as axially loaded using one of the effective slenderness ratios specified in Section E5(a) or E5(b), provided that the following requirements are met:

(1) Members are loaded at the ends in compression through the same one leg.
(2) Members are attached by welding or by connections with a minimum of two bolts.
(3) There are no intermediate transverse loads.
(4) L_c/r as determined in this section does not exceed 200.
(5) For unequal leg angles, the ratio of long leg width to short leg width is less than 1.7.

Single-angle members that do not meet these requirements or the requirements described in Section E5(a) or (b) shall be evaluated for combined axial load and flexure using the provisions of Chapter H.

(a) For angles that are individual members or are web members of planar trusses with adjacent web members attached to the same side of the gusset plate or chord

(1) For equal-leg angles or unequal-leg angles connected through the longer leg

(i) When $\frac{L}{r_a} \le 80$

$$\frac{L_c}{r} = 72 + 0.75\frac{L}{r_a} \quad \text{(E5-1)}$$

(ii) When $\frac{L}{r_a} > 80$

$$\frac{L_c}{r} = 32 + 1.25\frac{L}{r_a} \quad \text{(E5-2)}$$

(2) For unequal-leg angles connected through the shorter leg, L_c/r from Equations E5-1 and E5-2 shall be increased by adding $4\left[\left(b_l/b_s\right)^2 - 1\right]$, but L_c/r of the members shall not be taken as less than $0.95L/r_z$.

(b) For angles that are web members of box or space trusses with adjacent web members attached to the same side of the gusset plate or chord

(1) For equal-leg angles or unequal-leg angles connected through the longer leg

(i) When $\frac{L}{r_a} \le 75$

$$\frac{L_c}{r} = 60 + 0.8\frac{L}{r_a} \quad \text{(E5-3)}$$

(ii) When $\frac{L}{r_a} > 75$

$$\frac{L_c}{r} = 45 + \frac{L}{r_a} \quad \text{(E5-4)}$$

(2) For unequal-leg angles with leg length ratios less than 1.7 and connected through the shorter leg, L_c/r from Equations E5-3 and E5-4 shall be increased by adding $6\left[\left(b_l/b_s\right)^2 - 1\right]$, but L_c/r of the member shall not be taken as less than $0.82L/r_z$

where

L = length of member between work points at truss chord centerlines, in. (mm)
L_c = effective length of the member for buckling about the minor axis, in. (mm)
b_l = length of longer leg of angle, in. (mm)
b_s = length of shorter leg of angle, in. (mm)
r_a = radius of gyration about the geometric axis parallel to the connected leg, in. (mm)
r_z = radius of gyration about the minor principal axis, in. (mm)

E6. BUILT-UP MEMBERS

1. Compressive Strength

This section applies to built-up members composed of two shapes either (a) interconnected by bolts or welds or (b) with at least one open side interconnected by perforated cover plates or lacing with tie plates. The end connection shall be welded or connected by means of pretensioned bolts with Class A or B faying surfaces.

User Note: It is acceptable to design a bolted end connection of a built-up compression member for the full compressive load with bolts in bearing and bolt design based on the shear strength; however, the bolts must be pretensioned. In built-up compression members, such as double-angle struts in trusses, a small relative slip between the elements can significantly reduce the compressive strength of the strut. Therefore, the connection between the elements at the ends of built-up members should be designed to resist slip.

The nominal compressive strength of built-up members composed of two shapes that are interconnected by bolts or welds shall be determined in accordance with Sections E3, E4, or E7, subject to the following modification. In lieu of more accurate analysis, if the buckling mode involves relative deformations that produce shear forces in the connectors between individual shapes, L_c/r is replaced by $(L_c/r)_m$, determined as follows:

(a) For intermediate connectors that are bolted snug-tight

$$\left(\frac{L_c}{r}\right)_m = \sqrt{\left(\frac{L_c}{r}\right)_o^2 + \left(\frac{a}{r_i}\right)^2} \qquad \text{(E6-1)}$$

(b) For intermediate connectors that are welded or are connected by means of pretensioned bolts with Class A or B faying surfaces

(1) When $\dfrac{a}{r_i} \leq 40$

$$\left(\frac{L_c}{r}\right)_m = \left(\frac{L_c}{r}\right)_o \qquad \text{(E6-2a)}$$

(2) When $\dfrac{a}{r_i} > 40$

$$\left(\frac{L_c}{r}\right)_m = \sqrt{\left(\frac{L_c}{r}\right)_o^2 + \left(\frac{K_i a}{r_i}\right)^2} \qquad \text{(E6-2b)}$$

where

$\left(\dfrac{L_c}{r}\right)_m$ = modified slenderness ratio of built-up member

$\left(\dfrac{L_c}{r}\right)_o$ = slenderness ratio of built-up member acting as a unit in the buckling direction being addressed

L_c = effective length of built-up member, in. (mm)

K_i = 0.50 for angles back-to-back
= 0.75 for channels back-to-back
= 0.86 for all other cases

a = distance between connectors, in. (mm)

r_i = minimum radius of gyration of individual component, in. (mm)

2. General Requirements

Built-up members shall meet the following requirements:

(a) Individual components of compression members composed of two or more shapes shall be connected to one another at intervals, a, such that the slenderness ratio, a/r_i , of each of the component shapes between the fasteners does not exceed three-fourths times the governing slenderness ratio of the built-up member. The minimum radius of gyration, r_i, shall be used in computing the slenderness ratio of each component part.

(b) At the ends of built-up compression members bearing on base plates or finished surfaces, all components in contact with one another shall be connected by a weld having a length not less than the maximum width of the member or by bolts spaced longitudinally not more than four diameters apart for a distance equal to 1-1/2 times the maximum width of the member.

Along the length of built-up compression members between the end connections required in the foregoing, longitudinal spacing of intermittent welds or bolts shall be adequate to provide the required strength. For limitations on the longitudinal spacing of fasteners between elements in continuous contact consisting of a plate and a shape, or two plates, see Section J3.5. Where a component of a built-up compression member consists of an outside plate, the maximum spacing shall not exceed the thickness of the thinner outside plate times $0.75\sqrt{E/F_y}$, nor 12 in. (300 mm), when intermittent welds are provided along the edges of the components or when fasteners are provided on all gage lines at each section. When fasteners are staggered, the maximum spacing of fasteners on each gage line shall not exceed the thickness of the thinner outside plate times $1.12\sqrt{E/F_y}$ nor 18 in. (460 mm).

(c) Open sides of compression members built up from plates or shapes shall be provided with continuous cover plates perforated with a succession of access openings. The unsupported width of such plates at access openings, as defined in Section B4.1, is assumed to contribute to the available strength provided the following requirements are met:

(1) The width-to-thickness ratio shall conform to the limitations of Section B4.1.

User Note: It is conservative to use the limiting width-to-thickness ratio for Case 7 in Table B4.1a with the width, b, taken as the transverse distance between the nearest lines of fasteners. The net area of the plate is taken at the widest hole. In lieu of this approach, the limiting width-to-thickness ratio may be determined through analysis.

(2) The ratio of length (in direction of stress) to width of hole shall not exceed 2.

(3) The clear distance between holes in the direction of stress shall be not less than the transverse distance between nearest lines of connecting fasteners or welds.

(4) The periphery of the holes at all points shall have a minimum radius of 1-1/2 in. (38 mm).

(d) As an alternative to perforated cover plates, lacing with tie plates is permitted at each end and at intermediate points if the lacing is interrupted. Tie plates shall be as near the ends as practicable. In members providing available strength, the end tie plates shall have a length of not less than the distance between the lines of fasteners or welds connecting them to the components of the member. Intermediate tie plates shall have a length not less than one-half of this distance. The thickness of tie plates shall be not less than one-fiftieth of the distance between lines of welds or fasteners connecting them to the segments of the members. In welded construction, the welding on each line connecting a tie plate shall total not less than one-third the length of the plate. In bolted construction, the spacing in the direction of stress in tie plates shall be not more than six diameters and the tie plates shall be connected to each segment by at least three fasteners.

(e) Lacing, including flat bars, angles, channels or other shapes employed as lacing, shall be so spaced that L/r of the flange element included between their connections shall not exceed three-fourths times the governing slenderness ratio for the member as a whole. Lacing shall be proportioned to provide a shearing strength normal to the axis of the member equal to 2% of the available compressive strength of the member. For lacing bars arranged in single systems, L/r shall not exceed 140. For double lacing, this ratio shall not exceed 200. Double lacing bars shall be joined at the intersections. For lacing bars in compression, L is permitted to be taken as the unsupported length of the lacing bar between welds or fasteners connecting it to the components of the built-up member for single lacing, and 70% of that distance for double lacing.

User Note: The inclination of lacing bars to the axis of the member shall preferably be not less than 60º for single lacing and 45º for double lacing. When the distance between the lines of welds or fasteners in the flanges is more than 15 in. (380 mm), the lacing should preferably be double or made of angles.

For additional spacing requirements, see Section J3.5.

E7. MEMBERS WITH SLENDER ELEMENTS

This section applies to slender-element compression members, as defined in Section B4.1 for elements in axial compression.

The nominal compressive strength, P_n, shall be the lowest value based on the applicable limit states of flexural buckling, torsional buckling, and flexural-torsional buckling in interaction with local buckling.

$$P_n = F_n A_e \tag{E7-1}$$

where

A_e = summation of the effective areas of the cross section based on reduced effective widths, b_e, d_e or h_e, or the area as given by Equations E7-6 or E7-7, in.^{2} (mm^2)

F_n = nominal stress determined in accordance with Section E3 or E4, ksi (MPa). For single angles, determine F_n in accordance with Section E3 only.

User Note: The effective area, A_e, may be determined by deducting from the gross area, A_g, the reduction in area of each slender element determined as $(b-b_e)t$.

1. Slender Element Members Excluding Round HSS

The effective width, b_e, (for tees, this is d_e; for webs, this is h_e) for slender elements is determined as follows:

(a) When $\lambda \le \lambda_r \sqrt{\dfrac{F_y}{F_n}}$

$$b_e = b \tag{E7-2}$$

(b) When $\lambda > \lambda_r \sqrt{\dfrac{F_y}{F_n}}$

$$b_e = b\left(1 - c_1\sqrt{\frac{F_{el}}{F_n}}\right)\sqrt{\frac{F_{el}}{F_n}} \tag{E7-3}$$

where

b = width of the element (for tees this is d; for webs this is h), in. (mm)

c_1 = effective width imperfection adjustment factor determined from Table E7.1

$$c_2 = \frac{1-\sqrt{1-4c_1}}{2c_1} \tag{E7-4}$$

λ = width-to-thickness ratio for the element as defined in Section B4.1

λ_r = limiting width-to-thickness ratio as defined in Table B4.1a

$$F_{el} = \left(c_2\frac{\lambda_r}{\lambda}\right)^2 F_y \tag{E7-5}$$

= elastic local buckling stress determined according to Equation E7-5 or an elastic local buckling analysis, ksi (MPa)

Table E7.1
Effective Width Imperfection Adjustment Factors, c_1 and c_2

Case	Slender Element	c_1	c_2
(a)	Stiffened elements except walls of square and rectangular HSS	0.18	1.31
(b)	Walls of square and rectangular HSS	0.20	1.38
(c)	All other elements	0.22	1.49

2. Round HSS

The effective area, A_e, is determined as follows:

(a) When $\dfrac{D}{t} \le 0.11\dfrac{E}{F_y}$

$$A_e = A_g \tag{E7-6}$$

(b) When $0.11\dfrac{E}{F_y} < \dfrac{D}{t} < 0.45\dfrac{E}{F_y}$

$$A_e = \left[\frac{0.038E}{F_y (D/t)} + \frac{2}{3} \right] A_g \tag{E7-7}$$

where

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

t = thickness of wall, in. (mm)

CHAPTER F

DESIGN OF MEMBERS FOR FLEXURE

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

The chapter is organized as follows:

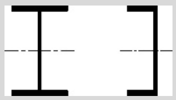
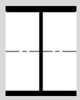
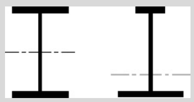
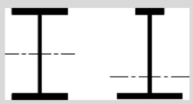
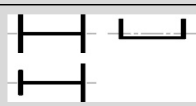
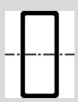
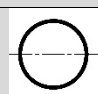
F1. General Provisions
F2. Doubly Symmetric Compact I-Shaped Members and Channels Bent about Their Major Axis
F3. Doubly Symmetric I-Shaped Members with Compact Webs and Noncompact or Slender Flanges Bent about Their Major Axis
F4. Other I-Shaped Members with Compact or Noncompact Webs Bent about Their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

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

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

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

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

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

Y = yielding, CFY = compression flange yielding, LTB = lateral-torsional buckling, FLB = flange local buckling, WLB = web local buckling, TFY = tension flange yielding, LLB = leg local buckling, LB = local buckling, C = compact, NC = noncompact, S = slender, N/A = not applicable

F1. GENERAL PROVISIONS

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

(a) For all provisions in this chapter

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

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

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

(c) For singly symmetric members in single curvature and all doubly symmetric members

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

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

where

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

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

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

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

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

For cantilevers where warping is prevented at the support and where the free end is unbraced, C_b = 1.0.

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

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

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

User Note: For F_y = 50 ksi (345 MPa), all current ASTM A6 W, S, M, C, and MC shapes except W21×48, W14×99, W14×90, W12×65, W10×12, W8×31, W8×10, W6×15, W6×9, W6×8.5, and M4×6 have compact flanges; For $F_y \leq$ 70 ksi (485 MPa), all current ASTM A6 W, S, M, HP, C, and MC shapes have compact webs.

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

1. Yielding

$$M_n = M_p = F_y Z_x \tag{F2-1}$$

where

F_y = specified minimum yield stress of the type of steel being used, ksi (MPa)

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

2. Lateral-Torsional Buckling

(a) When $L_b \leq L_p$, the limit state of lateral-torsional buckling does not apply.

(b) When $L_p < L_b \leq L_r$

$$M_n = C_b\left[M_p - \left(M_p - 0.7F_y S_x\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \leq M_p \tag{F2-2}$$

(c) When $L_b > L_r$

$$M_n = F_{cr} S_x \leq M_p \tag{F2-3}$$

where

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

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

= critical stress, ksi (MPa)

E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)

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

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

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

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

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

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

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

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

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

L_r, the limiting unbraced length for the limit state of inelastic lateral-torsional buckling, in. (mm), is:

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

where

r_y = radius of gyration about y-axis, in. (mm)

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

and the coefficient c is determined as follows:

(1) For doubly symmetric I-shapes

$$c = 1 \quad \text{(F2-8a)}$$

(2) For channels

$$c = \frac{h_o}{2}\sqrt{\frac{I_y}{C_w}} \quad \text{(F2-8b)}$$

where

I_y = moment of inertia about the y-axis, in.^{4} (mm^4)

User Note:

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

$$r_{ts}^2 = \frac{I_y h_o}{2S_x}$$

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

$$r_{ts} = \frac{b_f}{\sqrt{12\left(1 + \frac{1}{6}\frac{h t_w}{b_f t_f}\right)}}$$

F3. DOUBLY SYMMETRIC I-SHAPED MEMBERS WITH COMPACT WEBS AND NONCOMPACT OR SLENDER FLANGES BENT ABOUT THEIR MAJOR AXIS

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

> **User Note:** The following shapes have noncompact flanges for $F_y = 50$ ksi (345 MPa): W21×48, W14×99, W14×90, W12×65, W10×12, W8×31, W8×10, W6×15, W6×9, W6×8.5, and M4×6. All other ASTM A6 W, S, and M shapes have compact flanges for $F_y \le 50$ ksi (345 MPa).

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

1. Lateral-Torsional Buckling

For lateral-torsional buckling, the provisions of Section F2.2 shall apply.

2. Compression Flange Local Buckling

(a) For sections with noncompact flanges

$$M_n = M_p - \left(M_p - 0.7F_yS_x\right)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \tag{F3-1}$$

(b) For sections with slender flanges

$$M_n = \frac{0.9Ek_cS_x}{\lambda^2} \tag{F3-2}$$

where

$k_c = \dfrac{4}{\sqrt{h/t_w}}$ and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes

h = distance as defined in Section B4.1b, in. (mm)

$\lambda = \dfrac{b_f}{2t_f}$

b_f = width of the flange, in. (mm)

t_f = thickness of the flange, in. (mm)

$\lambda_{pf} = \lambda_p$ is the limiting width-to-thickness ratio for a compact flange as defined in Table B4.1b

$\lambda_{rf} = \lambda_r$ is the limiting width-to-thickness ratio for a noncompact flange as defined in Table B4.1b

F4. OTHER I-SHAPED MEMBERS WITH COMPACT OR NONCOMPACT WEBS BENT ABOUT THEIR MAJOR AXIS

This section applies to doubly symmetric I-shaped members bent about their major axis with noncompact webs and singly symmetric I-shaped members with webs attached to the mid-width of the flanges, bent about their major axis, with compact or noncompact webs, as defined in Section B4.1 for flexure.

User Note: I-shaped members for which this section is applicable may be designed conservatively using Section F5.

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

1. Compression Flange Yielding

$$M_n = R_{pc} M_{yc} \tag{F4-1}$$

where

M_{yc} = $F_y S_{xc}$ = yield moment in the compression flange, kip-in. (N-mm)
R_{pc} = web plastification factor, determined in accordance with Section F4.2(c)(6)
S_{xc} = elastic section modulus referred to compression flange, in.^{3} (mm^3)

2. Lateral-Torsional Buckling

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

(b) When $L_p < L_b \le L_r$

$$M_n = C_b \left[R_{pc} M_{yc} - \left(R_{pc} M_{yc} - F_L S_{xc} \right) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \le R_{pc} M_{yc} \tag{F4-2}$$

(c) When $L_b > L_r$

$$M_n = F_{cr} S_{xc} \le R_{pc} M_{yc} \tag{F4-3}$$

where

(1) M_{yc}, the yield moment in the compression flange, kip-in. (N-mm), is:

$$M_{yc} = F_y S_{xc} \tag{F4-4}$$

(2) F_{cr}, the critical stress, ksi (MPa), is:

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_t} \right)^2} \sqrt{1 + 0.078 \frac{J}{S_{xc} h_o} \left(\frac{L_b}{r_t} \right)^2} \tag{F4-5}$$

For $\frac{I_{yc}}{I_y} \le 0.23$, J shall be taken as zero,

where

I_{yc} = moment of inertia of the compression flange about the y-axis, in.^{4} (mm^4)

(3) F_L, nominal compression flange stress above which the inelastic buckling limit states apply, ksi (MPa), is determined as follows:

(i) When $\dfrac{S_{xt}}{S_{xc}} \geq 0.7$

$$F_L = 0.7F_y \qquad \text{(F4-6a)}$$

(ii) When $\dfrac{S_{xt}}{S_{xc}} < 0.7$

$$F_L = F_y \frac{S_{xt}}{S_{xc}} \geq 0.5F_y \qquad \text{(F4-6b)}$$

where

S_{xt} = elastic section modulus referred to tension flange, in.^{3} (mm^3)

(4) L_p, the limiting laterally unbraced length for the limit state of yieling, in. (mm) is:

$$L_p = 1.1r_t\sqrt{\frac{E}{F_y}} \qquad \text{(F4-7)}$$

(5) L_r, the limiting unbraced length for the limit state of inelastic lateral-torsional buckling, in. (mm), is:

$$L_r = 1.95r_t\frac{E}{F_L}\sqrt{\frac{J}{S_{xc}h_o} + \sqrt{\left(\frac{J}{S_{xc}h_o}\right)^2 + 6.76\left(\frac{F_L}{E}\right)^2}} \qquad \text{(F4-8)}$$

(6) R_{pc}, the web plastification factor, is determined as follows:

(i) When $I_{yc}/I_y > 0.23$

(a) When $\dfrac{h_c}{t_w} \leq \lambda_{pw}$

$$R_{pc} = \frac{M_p}{M_{yc}} \qquad \text{(F4-9a)}$$

(b) When $\dfrac{h_c}{t_w} > \lambda_{pw}$

$$R_{pc} = \left[\frac{M_p}{M_{yc}} - \left(\frac{M_p}{M_{yc}} - 1\right)\left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right)\right] \leq \frac{M_p}{M_{yc}} \qquad \text{(F4-9b)}$$

(ii) When $I_{yc}/I_y \leq 0.23$

$$R_{pc} = 1.0 \qquad \text{(F4-10)}$$

where

M_p = $F_yZ_x \leq 1.6F_yS_x$

h_c = twice the distance from the centroid to the following: the inside face of the compression flange less the fillet or corner radius, for rolled shapes; the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used, for built-up sections, in. (mm)

$$= \frac{h_c}{t_w}$$

λ_{pw} = λ_p, the limiting width-to-thickness ratio for a compact web as defined in Table B4.1b

λ_{rw} = λ_r, the limiting width-to-thickness ratio for a noncompact web as defined in Table B4.1b

(7) r_t, the effective radius of gyration for lateral-torsional buckling, in. (mm), is determined as follows:

(i) For I-shapes with a rectangular compression flange

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1+\frac{1}{6}a_w\right)}} \quad \text{(F4-11)}$$

where

$$a_w = \frac{h_c t_w}{b_{fc} t_{fc}} \quad \text{(F4-12)}$$

b_{fc} = width of compression flange, in. (mm)
t_{fc} = thickness of compression flange, in. (mm)
t_w = thickness of web, in. (mm)

(ii) For I-shapes with a channel cap or a cover plate attached to the compression flange

r_t = radius of gyration of the flange components in flexural compression plus one-third of the web area in compression due to application of major axis bending moment alone, in. (mm)

3. Compression Flange Local Buckling

(a) For sections with compact flanges, the limit state of local buckling does not apply.

(b) For sections with noncompact flanges

$$M_n = R_{pc}M_{yc} - \left(R_{pc}M_{yc} - F_L S_{xc}\right)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \quad \text{(F4-13)}$$

(c) For sections with slender flanges

$$M_n = \frac{0.9Ek_c S_{xc}}{\lambda^2} \quad \text{(F4-14)}$$

where

F_L is defined in Equations F4-6a and F4-6b

R_{pc} is the web plastification factor, determined by Equation F4-9a, F4-9b, or F4-10

$k_c = \frac{4}{\sqrt{h/t_w}}$ and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes

$\lambda = \frac{b_{fc}}{2t_{fc}}$

λ_{pf} = λ_p, the limiting width-to-thickness ratio for a compact flange as defined in Table B4.1b

λ_{rf} = λ_r, the limiting width-to-thickness ratio for a noncompact flange as defined in Table B4.1b

4. Tension Flange Yielding

(a) When $S_{xt} \geq S_{xc}$, the limit state of tension flange yielding does not apply.

(b) When $S_{xt} < S_{xc}$

$$M_n = R_{pt} M_{yt} \quad \text{(F4-15)}$$

where

$M_{yt} = F_y S_{xt}$ = yield moment in the tension flange, kip-in. (N-mm)

R_{pt}, the web plastification factor corresponding to the tension flange yielding limit state, is determined as follows:

(1) When $I_{yc}/I_y > 0.23$

(i) When $\dfrac{h_c}{t_w} \leq \lambda_{pw}$

$$R_{pt} = \frac{M_p}{M_{yt}} \quad \text{(F4-16a)}$$

(ii) When $\dfrac{h_c}{t_w} > \lambda_{pw}$

$$R_{pt} = \left[\frac{M_p}{M_{yt}} - \left(\frac{M_p}{M_{yt}} - 1\right)\left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right)\right] \leq \frac{M_p}{M_{yt}} \quad \text{(F4-16b)}$$

(2) When $I_{yc}/I_y \leq 0.23$

$$R_{pt} = 1.0 \quad \text{(F4-17)}$$

where

$M_p = F_y Z_x \leq 1.6 F_y S_x$

$\lambda = \dfrac{h_c}{t_w}$

λ_{pw} = λ_p, the limiting width-to-thickness ratio for a compact web as defined in Table B4.1b

λ_{rw} = λ_r, the limiting width-to-thickness ratio for a noncompact web as defined in Table B4.1b

F5. DOUBLY SYMMETRIC AND SINGLY SYMMETRIC I-SHAPED MEMBERS WITH SLENDER WEBS BENT ABOUT THEIR MAJOR AXIS

This section applies to doubly symmetric and singly symmetric I-shaped members with slender webs attached to the mid-width of the flanges and bent about their major axis as defined in Section B4.1 for flexure.

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

1. Compression Flange Yielding

$$M_n = R_{pg} F_y S_{xc} \tag{F5-1}$$

2. Lateral-Torsional Buckling

$$M_n = R_{pg} F_{cr} S_{xc} \tag{F5-2}$$

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

(b) When $L_p < L_b \le L_r$

$$F_{cr} = C_b \left[F_y - (0.3F_y)\left(\frac{L_b - L_p}{L_r - L_p}\right) \right] \le F_y \tag{F5-3}$$

(c) When $L_b > L_r$

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\dfrac{L_b}{r_t}\right)^2} \le F_y \tag{F5-4}$$

where

L_p is defined by Equation F4-7

$$L_r = \pi r_t \sqrt{\frac{E}{0.7F_y}} \tag{F5-5}$$

r_t = effective radius of gyration for lateral-torsional buckling as defined in Section F4, in. (mm)

R_{pg}, the bending strength reduction factor, is:

$$R_{pg} = 1 - \frac{a_w}{1{,}200 + 300a_w}\left(\frac{h_c}{t_w} - 5.7\sqrt{\frac{E}{F_y}}\right) \le 1.0 \tag{F5-6}$$

and

a_w is defined by Equation F4-12, but shall not exceed 10

3. Compression Flange Local Buckling

$$M_n = R_{pg} F_{cr} S_{xc} \tag{F5-7}$$

(a) For sections with compact flanges, the limit state of compression flange local buckling does not apply.

(b) For sections with noncompact flanges

$$F_{cr} = F_y - (0.3F_y)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \tag{F5-8}$$

(c) For sections with slender flanges

$$F_{cr} = \frac{0.9Ek_c}{\left(\frac{b_f}{2t_f}\right)^2} \quad \text{(F5-9)}$$

where

$k_c = \dfrac{4}{\sqrt{h/t_w}}$ and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes

$\lambda = \dfrac{b_{fc}}{2t_{fc}}$

$\lambda_{pf} = \lambda_p$, the limiting width-to-thickness ratio for a compact flange as defined in Table B4.1b

$\lambda_{rf} = \lambda_r$, the limiting width-to-thickness ratio for a noncompact flange as defined in Table B4.1b

4. Tension Flange Yielding

(a) When $S_{xt} \geq S_{xc}$, the limit state of tension flange yielding does not apply.

(b) When $S_{xt} < S_{xc}$

$$M_n = F_y S_{xt} \quad \text{(F5-10)}$$

F6. I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MINOR AXIS

This section applies to I-shaped members and channels bent about their minor axis.

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

1. Yielding

$$M_n = M_p = F_y Z_y \leq 1.6 F_y S_y \quad \text{(F6-1)}$$

where

S_y = elastic section modulus taken about the y-axis, in.^{3} (mm^3)

Z_y = plastic section modulus taken about the y-axis, in.^{3} (mm^3)

2. Flange Local Buckling

(a) For sections with compact flanges, the limit state of flange local buckling does not apply.

> **User Note:** For F_y = 50 ksi (345 MPa), all current ASTM A6 W, S, M, C, and MC shapes except W21x48, W14x99, W14x90, W12x65, W10x12, W8x31, W8x10, W6x15, W6x9, W6x8.5, and M4x6 have compact flanges.

(b) For sections with noncompact flanges

$$M_n = M_p - \left(M_p - 0.7F_y S_y\right)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \quad \text{(F6-2)}$$

(c) For sections with slender flanges

$$M_n = F_{cr} S_y \quad \text{(F6-3)}$$

where

$$F_{cr} = \frac{0.70E}{\left(\dfrac{b}{t_f}\right)^2} \quad \text{(F6-4)}$$

b = for flanges of I-shaped members, half the full flange width, b_f; for flanges of channels, the full nominal dimension of the flange, in. (mm)

t_f = thickness of the flange, in. (mm)

$\lambda = \dfrac{b}{t_f}$

$\lambda_{pf} = \lambda_p$, the limiting width-to-thickness ratio for a compact flange as defined in Table B4.1b

$\lambda_{rf} = \lambda_r$, the limiting width-to-thickness ratio for a noncompact flange as defined in Table B4.1b

F7. SQUARE AND RECTANGULAR HSS AND BOX SECTIONS

This section applies to square and rectangular HSS, and box sections bent about either axis, having compact, noncompact, or slender webs or flanges, as defined in Section B4.1 for flexure.

The nominal flexural strength, M_n, shall be the lowest value obtained according to the limit states of yielding (plastic moment), flange local buckling, web local buckling, and lateral-torsional buckling under pure flexure.

1. Yielding

$$M_n = M_p = F_y Z \quad \text{(F7-1)}$$

where

Z = plastic section modulus about the axis of bending, in.^{3} (mm^3)

2. Flange Local Buckling

(a) For compact sections, the limit state of flange local buckling does not apply.

(b) For sections with noncompact flanges

$$M_n = M_p - \left(M_p - F_y S\right)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \le M_p \quad \text{(F7-2)}$$

where

S = elastic section modulus about the axis of bending, in.^{3} (mm^3)

b = width of compression flange as defined in Section B4.1b, in. (mm)

t_f = thickness of the flange, in. (mm)

$\lambda = \dfrac{b}{t_f}$

λ_{pf} = λ_p, the limiting width-to-thickness ratio for a compact flange as defined in Table B4.1b

λ_{rf} = λ_r, the limiting width-to-thickness ratio for a noncompact flange as defined in Table B4.1b

(c) For sections with slender flanges

$$M_n = F_y S_e \tag{F7-3}$$

where

S_e = effective section modulus determined with the effective width, b_e, of the compression flange taken as:

(1) For HSS

$$b_e = 1.92 t_f \sqrt{\frac{E}{F_y}}\left(1 - \frac{0.38}{b/t_f}\sqrt{\frac{E}{F_y}}\right) \le b \tag{F7-4}$$

(2) For box sections

$$b_e = 1.92 t_f \sqrt{\frac{E}{F_y}}\left(1 - \frac{0.34}{b/t_f}\sqrt{\frac{E}{F_y}}\right) \le b \tag{F7-5}$$

3. Web Local Buckling

(a) For compact sections, the limit state of web local buckling does not apply.

(b) For sections with noncompact webs

$$M_n = M_p - \left(M_p - F_y S\right)\left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right) \le M_p \tag{F7-6}$$

where

h = depth of web, as defined in Section B4.1b, in. (mm)

t_w = thickness of the web, in. (mm)

$\lambda = \dfrac{h}{t_w}$

λ_{pw} = λ_p, the limiting width-to-thickness ratio for a compact web as defined in Table B4.1b

λ_{rw} = λ_r, the limiting width-to-thickness ratio for a noncompact web as defined in Table B4.1b

(c) For sections with slender webs and compact or noncompact flanges

$$M_n = R_{pg} F_y S \tag{F7-7}$$

where

R_{pg} is defined by Equation F5-6 with $a_w = 2ht_w / \left(bt_f\right)$

> **User Note:** Box sections with slender webs and slender flanges are not addressed in this Specification.

> **User Note:** There are no HSS with slender webs.

4. Lateral-Torsional Buckling

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

(b) When $L_p < L_b \le L_r$

$$M_n = C_b\left[M_p - \left(M_p - 0.7F_yS_x\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p \qquad \text{(F7-10)}$$

(c) When $L_b > L_r$

$$M_n = 2EC_b\frac{\sqrt{JA_g}}{L_b/r_y} \le M_p \qquad \text{(F7-11)}$$

where

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

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

$$L_p = 0.13Er_y\frac{\sqrt{JA_g}}{M_p} \qquad \text{(F7-12)}$$

L_r, the limiting laterally unbraced length for the limit state of inelastic lateral-torsional buckling, in. (mm), is:

$$L_r = 2Er_y\frac{\sqrt{JA_g}}{0.7F_yS_x} \qquad \text{(F7-13)}$$

> **User Note:** Lateral-torsional buckling will not occur in square sections or sections bending about their minor axis. In HSS sizes, deflection will usually control before there is a significant reduction in flexural strength due to lateral-torsional buckling. The same is true for box sections, and lateral-torsional buckling will usually only be a consideration for sections with high depth-to-width ratios.

F8. ROUND HSS

This section applies to round HSS having D/t ratios of less than $\frac{0.45E}{F_y}$.

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

1. Yielding

$$M_n = M_p = F_yZ \qquad \text{(F8-1)}$$

2. Local Buckling

(a) For compact sections, the limit state of flange local buckling does not apply.

(b) For noncompact sections

$$M_n = \left[\frac{0.021E}{\frac{D}{t}} + F_y\right]S \tag{F8-2}$$

(c) For sections with slender walls

$$M_n = F_{cr}S \tag{F8-3}$$

where

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

$$F_{cr} = \frac{0.33E}{\left(\frac{D}{t}\right)} \tag{F8-4}$$

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

F9. TEES AND DOUBLE ANGLES LOADED IN THE PLANE OF SYMMETRY

This section applies to tees and double angles loaded in the plane of symmetry.

The nominal flexural strength, M_n, shall be the lowest value obtained according to the limit states of yielding (plastic moment), lateral-torsional buckling, flange local buckling, and local buckling of tee stems and double angle web legs.

1. Yielding

$$M_n = M_p \tag{F9-1}$$

where

(a) For tee stems and web legs in tension

$$M_p = F_y Z_x \le 1.6M_y \tag{F9-2}$$

where

M_y = yield moment about the axis of bending, kip-in. (N-mm)

$= F_y S_x$ (F9-3)

(b) For tee stems in compression

$$M_p = M_y \tag{F9-4}$$

(c) For double angles with web legs in compression

$$M_p = 1.5M_y \tag{F9-5}$$

2. Lateral-Torsional Buckling

(a) For stems and web legs in tension

(1) When $L_b \le L_p$, the limit state of lateral-torsional buckling does not apply.

(2) When $L_p < L_b \le L_r$

$$M_n = M_p - \left(M_p - M_y\right)\left(\frac{L_b - L_p}{L_r - L_p}\right) \quad \text{(F9-6)}$$

(3) When $L_b > L_r$

$$M_n = M_{cr} \quad \text{(F9-7)}$$

where

$$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} \quad \text{(F9-8)}$$

$$L_r = 1.95\left(\frac{E}{F_y}\right)\frac{\sqrt{I_y J}}{S_x}\sqrt{2.36\left(\frac{F_y}{E}\right)\frac{dS_x}{J}+1} \quad \text{(F9-9)}$$

$$M_{cr} = \frac{1.95E}{L_b}\sqrt{I_y J}\left(B+\sqrt{1+B^2}\right) \quad \text{(F9-10)}$$

$$B = 2.3\left(\frac{d}{L_b}\right)\sqrt{\frac{I_y}{J}} \quad \text{(F9-11)}$$

d = depth of tee or width of web leg in tension, in. (mm)

(b) For stems and web legs in compression anywhere along the unbraced length, M_{cr} is given by Equation F9-10 with

$$B = -2.3\left(\frac{d}{L_b}\right)\sqrt{\frac{I_y}{J}} \quad \text{(F9-12)}$$

where

d = depth of tee or width of web leg in compression, in. (mm)

(1) For tee stems

$$M_n = M_{cr} \le M_y \quad \text{(F9-13)}$$

(2) For double-angle web legs, M_n shall be determined using Equations F10-2 and F10-3 with M_{cr} determined using Equation F9-10 and M_y determined using Equation F9-3.

3. Flange Local Buckling of Tees and Double-Angle Legs

(a) For tee flanges

(1) For sections with a compact flange in flexural compression, the limit state of flange local buckling does not apply.

(2) For sections with a noncompact flange in flexural compression

$$M_n = \left[M_p - \left(M_p - 0.7F_y S_{xc}\right)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] \le 1.6M_y \quad \text{(F9-14)}$$

(3) For sections with a slender flange in flexural compression

$$M_n = \frac{0.7ES_{xc}}{\left(\dfrac{b_f}{2t_f}\right)^2} \quad \text{(F9-15)}$$

where

S_{xc} = elastic section modulus referred to the compression flange, in.^{3} (mm^3)

$\lambda = \dfrac{b_f}{2t_f}$

$\lambda_{pf} = \lambda_p$, the limiting width-to-thickness ratio for a compact flange as defined in Table B4.1b

$\lambda_{rf} = \lambda_r$, the limiting width-to-thickness ratio for a noncompact flange as defined in Table B4.1b

(b) For double-angle flange legs

The nominal flexural strength, M_n, for double angles with the flange legs in compression shall be determined in accordance with Section F10.3, with S_c referred to the compression flange.

4. Local Buckling of Tee Stems and Double-Angle Web Legs in Flexural Compression

(a) For tee stems

$$M_n = F_{cr} S_x \quad \text{(F9-16)}$$

where

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

F_{cr}, the critical stress, is determined as follows:

(1) When $\dfrac{d}{t_w} \leq 0.84\sqrt{\dfrac{E}{F_y}}$

$$F_{cr} = F_y \quad \text{(F9-17)}$$

(2) When $0.84\sqrt{\dfrac{E}{F_y}} < \dfrac{d}{t_w} \leq 1.52\sqrt{\dfrac{E}{F_y}}$

$$F_{cr} = \left(1.43 - 0.515\frac{d}{t_w}\sqrt{\frac{F_y}{E}}\right)F_y \quad \text{(F9-18)}$$

(3) When $\dfrac{d}{t_w} > 1.52\sqrt{\dfrac{E}{F_y}}$

$$F_{cr} = \frac{1.52E}{\left(\frac{d}{t_w}\right)^2} \quad \text{(F9-19)}$$

(b) For double-angle web legs

The nominal flexural strength, M_n, for double angles with the web legs in compression shall be determined in accordance with Section F10.3, with S_c taken as the elastic section modulus.

F10. SINGLE ANGLES

This section applies to single angles with and without continuous lateral restraint along their length.

Single angles with continuous lateral-torsional restraint along the length are permitted to be designed on the basis of geometric axis (x, y) bending. Single angles without continuous lateral-torsional restraint along the length shall be designed using the provisions for principal axis bending except where the provision for bending about a geometric axis is permitted.

If the moment resultant has components about both principal axes, with or without axial load, or the moment is about one principal axis and there is axial load, the combined stress ratio shall be determined using the provisions of Section H2.

User Note: For geometric axis design, use section properties computed about the *x*- and *y*-axis of the angle, parallel and perpendicular to the legs. For principal axis design, use section properties computed about the major and minor principal axes of the angle.

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

User Note: For bending about the minor principal axis, only the limit states of yielding and leg local buckling apply.

1. Yielding

$$M_n = 1.5M_y \quad \text{(F10-1)}$$

2. Lateral-Torsional Buckling

For single angles without continuous lateral-torsional restraint along the length

(a) When $\frac{M_y}{M_{cr}} \leq 1.0$

$$M_n = \left(1.92 - 1.17\sqrt{\frac{M_y}{M_{cr}}}\right)M_y \leq 1.5M_y \quad \text{(F10-2)}$$

(b) When $\frac{M_y}{M_{cr}} > 1.0$

$$M_n = \left(0.92 - \frac{0.17M_{cr}}{M_y}\right)M_{cr} \quad \text{(F10-3)}$$

where

M_{cr}, the elastic lateral-torsional buckling moment, is determined as follows:

(1) For bending about the major principal axis of single angles

$$M_{cr} = \frac{9EA_g r_z tC_b}{8L_b}\left[\sqrt{1+\left(4.4\frac{\beta_w r_z}{L_b t}\right)^2} + 4.4\frac{\beta_w r_z}{L_b t}\right] \quad \text{(F10-4)}$$

where

C_b is computed using Equation F1-1 with a maximum value of 1.5

A_g = gross area of angle, in.^{2} (mm^2)

L_b = laterally unbraced length of member, in. (mm)

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

t = thickness of angle leg, in. (mm)

β_w = section property for single angles about major principal axis, in. (mm). β_w is positive with short legs in compression and negative with long legs in compression for unequal-leg angles, and zero for equal-leg angles. If the long leg is in compression anywhere along the unbraced length of the member, the negative value of β_w shall be used.

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

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

(i) With no lateral-torsional restraint:

(a) With maximum compression at the toe

$$M_{cr} = \frac{0.58Eb^4tC_b}{L_b^2}\left[\sqrt{1+0.88\left(\frac{L_b t}{b^2}\right)^2} - 1\right] \quad \text{(F10-5a)}$$

(b) With maximum tension at the toe

$$M_{cr} = \frac{0.58Eb^4tC_b}{L_b^2}\left[\sqrt{1+0.88\left(\frac{L_b t}{b^2}\right)^2} + 1\right] \quad \text{(F10-5b)}$$

where

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

b = width of leg, in. (mm)

(ii) With lateral-torsional restraint at the point of maximum moment only:

M_{cr} shall be taken as 1.25 times M_{cr} computed using Equation F10-5a or F10-5b.

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

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

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

3. Leg Local Buckling

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

(a) For compact sections, the limit state of leg local buckling does not apply.

(b) For sections with noncompact legs

$$M_n = F_y S_c\left[2.43 - 1.72\left(\frac{b}{t}\right)\sqrt{\frac{F_y}{E}}\right] \quad \text{(F10-6)}$$

(c) For sections with slender legs

$$M_n = F_{cr} S_c \quad \text{(F10-7)}$$

where

$$F_{cr} = \frac{0.71E}{\left(\frac{b}{t}\right)^2} \quad \text{(F10-8)}$$

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

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

F11. RECTANGULAR BARS AND ROUNDS

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

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

1. Yielding

For rectangular bars

$$M_n = M_p = F_y Z \le 1.5 F_y S_x \quad \text{(F11-1)}$$

For rounds

$$M_n = M_p = F_y Z \le 1.6 F_y S_x \quad \text{(F11-2)}$$

2. Lateral-Torsional Buckling

(a) For rectangular bars with $\frac{L_b d}{t^2} \le \frac{0.08E}{F_y}$ bent about their major axis, rectangular bars bent about their minor axis, and rounds, the limit state of lateral-torsional buckling does not apply.

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

$$M_n = C_b \left[1.52 - 0.274\left(\frac{L_b d}{t^2}\right)\frac{F_y}{E}\right] M_y \le M_p \qquad \text{(F11-3)}$$

where

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

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

$$M_n = F_{cr} S_x \le M_p \qquad \text{(F11-4)}$$

where

$$F_{cr} = \frac{1.9EC_b}{\frac{L_b d}{t^2}} \qquad \text{(F11-5)}$$

F12. UNSYMMETRICAL SHAPES

This section applies to all unsymmetrical shapes except single angles.

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

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

where

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

User Note: The design provisions within this section can be overly conservative for certain shapes, unbraced lengths, and moment diagrams. To improve economy, the provisions of Appendix 1.3 are recommended as an alternative for determining the nominal flexural strength of members of unsymmetrical shape.

1. Yielding

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

2. Lateral-Torsional Buckling

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

where

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

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

3. Local Buckling

$$F_n = F_{cr} \le F_y \tag{F12-4}$$

where

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

F13. PROPORTIONS OF BEAMS AND GIRDERS

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

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

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

(a) When $F_u A_{fn} \ge Y_t F_y A_{fg}$, the limit state of tensile rupture does not apply.

(b) When $F_u A_{fn} < Y_t F_y A_{fg}$, the nominal flexural strength, M_n, at the location of the holes in the tension flange shall not be taken greater than

$$M_n = \frac{F_u A_{fn}}{A_{fg}} S_x \tag{F13-1}$$

where

A_{fg} = gross area of tension flange, calculated in accordance with Section B4.3a, in.^{2} (mm^2)

A_{fn} = net area of tension flange, calculated in accordance with Section B4.3b, in.^{2} (mm^2)

F_u = specified minimum tensile strength, ksi (MPa)

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

Y_t = 1.0 for $F_y/F_u \le 0.8$
= 1.1 otherwise

2. Proportioning Limits for I-Shaped Members

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

$$0.1 \le \frac{I_{yc}}{I_y} \le 0.9 \tag{F13-2}$$

I-shaped members with slender webs shall also satisfy the following limits:

(a) When $\dfrac{a}{h} \le 1.5$

$$\left(\frac{h}{t_w}\right)_{max} = 12.0\sqrt{\frac{E}{F_y}} \quad \text{(F13-3)}$$

(b) When $\frac{a}{h} > 1.5$

$$\left(\frac{h}{t_w}\right)_{max} = \frac{0.40E}{F_y} \quad \text{(F13-4)}$$

where

a = clear distance between transverse stiffeners, in. (mm)

In unstiffened girders, h/t_w shall not exceed 260. The ratio of 2 times the web area in compression to the compression flange area, a_w, as defined by Equation F4-12, shall not exceed 10.

3. Cover Plates

For members with cover plates, the following provisions apply:

(a) Flanges of welded beams or girders are permitted to be varied in thickness or width by splicing a series of plates or by the use of cover plates.

(b) High-strength bolts or welds connecting flange to web, or cover plate to flange, shall be proportioned to resist the total horizontal shear resulting from the bending forces on the girder. The longitudinal distribution of these bolts or intermittent welds shall be in proportion to the intensity of the shear.

(c) However, the longitudinal spacing shall not exceed the maximum specified for compression or tension members in Sections E6 or D4, respectively. Bolts or welds connecting flange to web shall also be proportioned to transmit to the web any loads applied directly to the flange, unless provision is made to transmit such loads by direct bearing.

(d) Partial-length cover plates shall be extended beyond the theoretical cutoff point and the extended portion shall be attached to the beam or girder by high-strength bolts in a slip-critical connection or fillet welds. The attachment shall, at the applicable strength given in Sections J2.2, J3.8 or B3.11, develop the cover plate’s portion of the flexural strength in the beam or girder at the theoretical cutoff point.

(e) For welded cover plates, the welds connecting the cover plate termination to the beam or girder shall be continuous welds along both edges of the cover plate in the length a', defined in the following, and shall develop the cover plate’s portion of the available strength of the beam or girder at the distance a' from the end of the cover plate.

(1) When there is a continuous weld equal to or larger than three-fourths of the plate thickness across the end of the plate

$$a' = w \quad \text{(F13-5)}$$

where

w = width of cover plate, in. (mm)

(2) When there is a continuous weld smaller than three-fourths of the plate thickness across the end of the plate

$$a' = 1.5w \tag{F13-6}$$

(3) When there is no weld across the end of the plate

$$a' = 2w \tag{F13-7}$$

4. Built-Up Beams

Where two or more beams or channels are used side by side to form a flexural member, they shall be connected together in compliance with Section E6.2. When concentrated loads are carried from one beam to another or distributed between the beams, diaphragms having sufficient stiffness to distribute the load shall be welded or bolted between the beams.

CHAPTER G

DESIGN OF MEMBERS FOR SHEAR

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

The chapter is organized as follows:

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

- H3.3 Unsymmetric sections
- J4.2 Shear strength of connecting elements
- J10.6 Web panel zone shear

G1. GENERAL PROVISIONS

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

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

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

(b) The nominal shear strength, V_n, shall be determined according to Sections G2 through G7.

G2. I-SHAPED MEMBERS AND CHANNELS

This section addresses the determination of shear strength for I-shaped members and channels. Section G2.1 is applicable for webs with and without transverse stiffeners. Alternatively, Sections G2.2 and G2.3 may be used for webs with transverse stiffeners. Transverse stiffeners, or components providing equivalent restraint of out-of-plane deformation of the web, shall be provided at the member ends and at supports.

1. Shear Strength of Webs without Tension Field Action

The nominal shear strength, V_n, is:

$$V_n = 0.6F_y A_w C_{v1} \tag{G2-1}$$

where

F_y = specified minimum yield stress of the type of steel being used, ksi (MPa)
A_w = area of web, the overall depth times the web thickness, dt_w, in.^{2} (mm^2)

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

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

and

$$C_{v1} = 1.0 \tag{G2-2}$$

where
E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)
h = clear distance between flanges less the fillet at each flange, in. (mm)
t_w = thickness of web, in. (mm)

User Note: All current ASTM A6 W, S, and HP shapes except W44x230, W40x149, W36x135, W33x118, W30x90, W24x55, W16x26, and W12x14 meet the criteria stated in Section G2.1(a) for F_y = 50 ksi (345 MPa).

(b) For all other I-shaped members and channels

(1) The web shear strength coefficient, C_{v1}, is determined as follows:

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

$$C_{v1} = 1.0 \tag{G2-3}$$

where
h = for built-up welded sections, the clear distance between flanges, in. (mm)
= for built-up bolted sections, the distance between fastener lines, in. (mm)

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

$$C_{v1} = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} \tag{G2-4}$$

(2) The web plate shear buckling coefficient, k_v, is determined as follows:

(i) For webs without transverse stiffeners

$$k_v = 5.34$$

(ii) For webs with transverse stiffeners

$$k_v = 5 + \frac{5}{(a/h)^2} \tag{G2-5}$$

$$= 5.34 \text{ when } a/h > 3.0$$

where

a = clear distance between transverse stiffeners, in. (mm)

User Note: C_{v1}= 1.0 for all ASTM A6 W, S, M, and HP shapes except M12.5x12.4, M12.5x11.6, M12x11.8, M12x10.8, M12x10, M10x8, and M10x7.5, when $F_y = 50$ ksi (345 MPa).

2. Shear Strength of Interior Web Panels with $a/h \le 3$ Considering Tension Field Action

The nominal shear strength, V_n, is determined as follows:

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

$$V_n = 0.6 F_y A_w \qquad \text{(G2-6)}$$

(b) When $h/t_w > 1.10\sqrt{k_v E / F_y}$

(1) When $2A_w/(A_{fc} + A_{ft}) \le 2.5$, $h/b_{fc} \le 6.0$ and $h/b_{ft} \le 6.0$

$$V_n = 0.6 F_y A_w \left[C_{v2} + \frac{1 - C_{v2}}{1.15\sqrt{1+(a/h)^2}} \right] \qquad \text{(G2-7)}$$

(2) Otherwise

$$V_n = 0.6 F_y A_w \left[C_{v2} + \frac{1 - C_{v2}}{1.15\left[a/h + \sqrt{1+(a/h)^2} \right]} \right] \qquad \text{(G2-8)}$$

where

The web shear buckling coefficient, C_{v2}, is determined as follows:

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

$$C_{v2} = 1.0 \qquad \text{(G2-9)}$$

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

$$C_{v2} = \frac{1.10\sqrt{k_v E / F_y}}{h/t_w} \qquad \text{(G2-10)}$$

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

$$C_{v2} = \frac{1.51 k_v E}{(h/t_w)^2 F_y} \qquad \text{(G2-11)}$$

A_{fc} = area of compression flange, in.^{2} (mm^2)
A_{ft} = area of tension flange, in.^{2} (mm^2)
b_{fc} = width of compression flange, in. (mm)
b_{ft} = width of tension flange, in. (mm)

k_v is as defined in Section G2.1

The nominal shear strength is permitted to be taken as the larger of the values from Sections G2.1 and G2.2.

User Note: Section G2.1 may predict a higher strength for members that do not meet the requirements of Section G2.2(b)(1).

3. Shear Strength of End Web Panels with *a*/*h* ≤ 3 Considering Tension Field Action

(a) The nominal shear strength for I-shaped members with equal flange areas in the end panel, V_n, is

$$V_n = 0.6F_{yw}A_w\left[C_{v2} + \beta_v\left(\frac{1-C_{v2}}{1.15\sqrt{1+(a/h)^2}}\right)\right] \quad \text{(G2-12)}$$

where

$$\beta_v = \frac{2.8\left(\sqrt{M_{pf}+M_{pm}} + \sqrt{M_{pst}+M_{pm}}\right)}{h\sqrt{F_{yw}t_w(1-C_{v2})}} \le 1.0 \quad \text{(G2-13)}$$

and

F_{yw} = specified minimum yield stress of the web material, ksi (MPa)

M_{pf} = plastic moment of a section composed of the flange and a segment of the web with the depth, d_e, kip-in. (N-mm)

M_{pm} = smaller of M_{pf} and M_{pst}, kip-in. (N-mm)

M_{pst} = plastic moment of a section composed of the stiffener plus a length of web equal to d_e plus the distance from the inside face of the stiffener to the end of the beam, except that the distance from the inside face of the stiffener to the end of the beam shall not exceed $0.84t_w\sqrt{E/F_y}$ for calculation purposes, kip-in. (N-mm)

(i) when $C_{v2} \le 0.8$

$$d_e = 35t_w(0.8-C_{v2})^2 \quad \text{(G2-14)}$$

(ii) when $C_{v2} > 0.8$

$$d_e = 0 \quad \text{(G2-15)}$$

(b) The flexural stress in the tension flange, $\alpha M_r/S_{xt}$, in the end panel shall not be larger than $0.35F_y$.

where

α =1.0 (LRFD); α = 1.6 (ASD)

(c) The nominal shear strength for I-shaped members with unequal flange areas shall be determined by analysis.

User Note: An approach for I-shaped members with unequal flange areas is discussed in the commentary.

4. Transverse Stiffeners

For transverse stiffeners, the following shall apply.

(a) Transverse stiffeners are not required where $h/t_w \le 2.54\sqrt{E/F_y}$, or where the available shear strength provided in accordance with Section G2.1 for k_v = 5.34 is greater than the required shear strength.

(b) Transverse stiffeners are permitted to be stopped short of the tension flange, provided bearing is not needed to transmit a concentrated load or reaction. The weld by which transverse stiffeners are attached to the web shall be terminated not less than four times nor more than six times the web thickness from the near toe of the web-to-flange weld or web-to-flange fillet. When single stiffeners are used, they shall be attached to the compression flange to resist any twist of the flange.

(c) Bolts connecting stiffeners to the girder web shall be spaced not more than 12 in. (300 mm) on center. If intermittent fillet welds are used, the clear distance between welds shall not be more than 16 times the web thickness nor more than 10 in. (250 mm).

(d) $$(b/t)_{st} \le 0.56\sqrt{\frac{E}{F_{yst}}} \tag{G2-16}$$

(e) $$I_{st} \ge I_{st2} + \left(I_{st1} - I_{st2}\right)\rho_w \tag{G2-17}$$

where

F_{yst} = specified minimum yield stress of the stiffener material, ksi (MPa)

I_{st} = moment of inertia of the transverse stiffeners about an axis in the web center for stiffener pairs, or about the face in contact with the web plate for single stiffeners, in.^{4} (mm^4)

$$I_{st1} = \frac{h^4 \rho_{st}^{1.3}}{40}\left(\frac{F_{yw}}{E}\right)^{1.5} \tag{G2-18}$$

= minimum moment of inertia of the transverse stiffeners required for development of the full shear post buckling resistance of the stiffened web panels, $V_r = V_{c1}$, in.^{4} (mm^4)

$$I_{st2} = \left[\frac{2.5}{\left(a/h\right)^2} - 2\right] b_p t_w^3 \ge 0.5 b_p t_w^3 \tag{G2-19}$$

= minimum moment of inertia of the transverse stiffeners required for development of the web shear buckling resistance, $V_r = V_{c2}$, in.^{4} (mm^4)

V_{c1} = available shear strength calculated with V_n as defined in Section G2.1 or G2.2, as applicable, kips (N)

V_{c2} = available shear strength, kips (N), calculated with $V_n = 0.6F_y A_w C_{v2}$

V_r = required shear strength in the panel being considered, kips (N)

b_p = smaller of the dimension a and h, in. (mm)

$(b/t)_{st}$ = width-to-thickness ratio of the stiffener

ρ_{st} = larger of F_{yw}/F_{yst} and 1.0

ρ_w = maximum shear ratio, $\left(\dfrac{V_r - V_{c2}}{V_{c1} - V_{c2}}\right) \geq 0,$ within the web panels on each side of the transverse stiffener

User Note: I_{st} may conservatively be taken as I_{st1}. Equation G2-19 provides the minimum stiffener moment of inertia required to attain the web shear post buckling resistance according to Sections G2.1 and G2.2, as applicable. If less post buckling shear strength is required, Equation G2-17 provides a linear interpolation between the minimum moment of inertia required to develop web shear buckling and that required to develop the web shear post buckling strength.

G3. SINGLE ANGLES AND TEES

The nominal shear strength, V_n, of a single-angle leg or a tee stem is:

$$V_n = 0.6F_y btC_{v2} \tag{G3-1}$$

where

C_{v2} = web shear buckling strength coefficient, as defined in Section G2.2 with $h/t_w = b/t$ and $k_v = 1.2$

b = width of the leg resisting the shear force or depth of the tee stem, in. (mm)

t = thickness of angle leg or tee stem, in. (mm)

G4. RECTANGULAR HSS, BOX SECTIONS, AND OTHER SINGLY AND DOUBLY SYMMETRIC MEMBERS

The nominal shear strength, V_n, is:

$$V_n = 0.6F_y A_w C_{v2} \tag{G4-1}$$

For rectangular HSS and box sections

A_w = $2ht$, in.^{2} (mm^2)

C_{v2} = web shear buckling strength coefficient, as defined in Section G2.2, with $h/t_w = h/t$ and $k_v = 5$

h = width resisting the shear force, taken as the clear distance between the flanges less the inside corner radius on each side for HSS or the clear distance between flanges for box sections, in. (mm). If the corner radius is not known, h shall be taken as the corresponding outside dimension minus 3 times the thickness.

t = design wall thickness, as defined in Section B4.2, in. (mm)

For other singly or doubly symmetric shapes

A_w = area of web or webs, taken as the sum of the overall depth times the web thickness, dt_w, in.^{2} (mm^2)

C_{v2} = web shear buckling strength coefficient, as defined in Section G2.2, with $h/t_w = h/t$ and $k_v = 5$

h = width resisting the shear force, in. (mm)

= for built-up welded sections, the clear distance between flanges, in. (mm)

= for built-up bolted sections, the distance between fastener lines, in. (mm)

t = web thickness, as defined in Section B4.2, in. (mm)

G5. ROUND HSS

The nominal shear strength, V_n, of round HSS, according to the limit states of shear yielding and shear buckling, shall be determined as:

$$V_n = F_{cr} A_g / 2 \tag{G5-1}$$

where

F_{cr} shall be the larger of

$$F_{cr} = \frac{1.60E}{\sqrt{\frac{L_v}{D}}\left(\frac{D}{t}\right)^{\frac{5}{4}}} \tag{G5-2a}$$

and

$$F_{cr} = \frac{0.78E}{\left(\frac{D}{t}\right)^{\frac{3}{2}}} \tag{G5-2b}$$

but shall not exceed $0.6F_y$

A_g = gross area of member, in.^{2} (mm^2)
D = outside diameter, in. (mm)
L_v = distance from maximum to zero shear force, in. (mm)
t = design wall thickness, in. (mm)

User Note: The shear buckling equations, Equations G5-2a and G5-2b, will control for D/t over 100, high-strength steels, and long lengths. For standard sections, shear yielding will usually control and $F_{cr} = 0.6F_y$.

G6. DOUBLY SYMMETRIC AND SINGLY SYMMETRIC MEMBERS SUBJECT TO MINOR-AXIS SHEAR

For doubly and singly symmetric members loaded in the minor axis without torsion, the nominal shear strength, V_n, for each shear resisting element is:

$$V_n = 0.6 F_y b_f t_f C_{v2} \tag{G6-1}$$

where

C_{v2} = web shear buckling strength coefficient, as defined in Section G2.2 with $h/t_w = b_f/2t_f$ for I-shaped members and tees, or $h/t_w = b_f/t_f$ for channels, and $k_v = 1.2$
b_f = width of flange, in. (mm)
t_f = thickness of flange, in. (mm)

User Note: C_{v2} = 1.0 for all ASTM A6 W, S, M, and HP shapes, when $F_y \le 70$ ksi (485 MPa).

G7. BEAMS AND GIRDERS WITH WEB OPENINGS

The effect of all web openings on the shear strength of steel and composite beams shall be determined. 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 members subject to torsion only.

The chapter is organized as follows:

H1. Doubly and Singly Symmetric Members Subject to Flexure and Axial Force
H2. Unsymmetric and Other Members Subject to Flexure and Axial Force
H3. Members Subject to Torsion and Combined Torsion, Flexure, Shear, and/or Axial Force
H4. Rupture of Flanges with Holes Subjected to Tension

User Note: For composite members, see Chapter I.

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

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

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

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

(a) When $\frac{P_r}{P_c} \geq 0.2$

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

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

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

where

P_r = required compressive strength, determined in accordance with Chapter C, using LRFD or ASD load combinations, kips (N)

P_c = available compressive strength, ϕP_n or P_n/Ω, determined in accordance with Chapter E, kips (N)

M_r = required flexural strength, determined in accordance with Chapter C, using LRFD or ASD load combinations, kip-in. (N-mm)

M_c = available flexural strength, ϕM_n or M_n/Ω, determined in accordance with Chapter F, kip-in. (N-mm)

x = subscript relating symbol to major axis bending

y = subscript relating symbol to minor axis bending

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

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

where

P_r = required tensile strength, determined in accordance with Chapter C, using LRFD or ASD load combinations, kips (N)

P_c = available tensile strength, ϕP_n or P_n/Ω, determined in accordance with Chapter D, kips (N)

For doubly symmetric members, C_b in Chapter F is permitted to be multiplied by $\sqrt{1+\dfrac{\alpha P_r}{P_{ey}}}$ when axial tension acts concurrently with flexure,

where

$$P_{ey} = \frac{\pi^2 EI_y}{L_b^2} \tag{H1-2}$$

$\alpha = 1.0$ (LRFD); $\alpha = 1.6$ (ASD)

and

E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)

I_y = moment of inertia about the y-axis, in.^{4} (mm^4)

L_b = length between points that are either braced against lateral displacement of the compression flange or braced against twist of the cross section, in.^{4} (mm^4)

3. Doubly Symmetric Rolled Compact Members Subject to Single-Axis Flexure and Compression

For doubly symmetric rolled compact members, with the effective length for torsional buckling less than or equal to the effective length for y-axis flexural buckling, $L_{cz} \le L_{cy}$, subjected to flexure and compression with moments primarily about their major axis, it is permissible to address the two independent limit states, in-plane instability and out-of-plane buckling or lateral-torsional buckling, separately in lieu of the combined approach provided in Section H1.1,

where

L_{cy} = effective length for buckling about the y-axis, in. (mm)

L_{cz} = effective length for buckling about the longitudinal axis, in. (mm)

For members with $M_{ry}/M_{cy} \ge 0.05$, the provisions of Section H1.1 shall be followed.

(a) For the limit state of in-plane instability, Equations H1-1a and H1-1b shall be used with P_c taken as the available compressive strength in the plane of bending and M_{cx} taken as the available flexural strength based on the limit state of yielding.

(b) For the limit state of out-of-plane buckling and lateral-torsional buckling

$$\frac{P_r}{P_{cy}}\left(1.5 - 0.5\frac{P_r}{P_{cy}}\right) + \left(\frac{M_{rx}}{C_b M_{cx}}\right)^2 \le 1.0 \tag{H1-3}$$

where

P_{cy} = available compressive strength out of the plane of bending, kips (N)

C_b = lateral-torsional buckling modification factor determined from Section F1

M_{cx} = available lateral-torsional strength for major axis flexure determined in accordance with Chapter F using $C_b = 1.0$, kip-in. (N-mm)

User Note: In Equation H1-3, $C_b M_{cx}$ may be larger than $\phi_b M_{px}$ in LRFD or M_{px}/Ω_b in ASD. The yielding resistance of the beam-column is captured by Equations H1-1a and H1-1b.

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.

$$\left| \frac{f_{ra}}{F_{ca}} + \frac{f_{rbw}}{F_{cbw}} + \frac{f_{rbz}}{F_{cbz}} \right| \le 1.0 \quad \text{(H2-1)}$$

where

f_{ra} = required axial stress at the point of consideration, determined in accordance with Chapter C, using LRFD or ASD load combinations, ksi (MPa)

F_{ca} = available axial stress at the point of consideration, determined in accordance with Chapter E for compression or Section D2 for tension, ksi (MPa)

f_{rbw}, f_{rbz} = required flexural stress at the point of consideration, determined in accordance with Chapter C, using LRFD or ASD load combinations, ksi (MPa).

F_{cbw}, F_{cbz} = available flexural stress at the point of consideration, determined in accordance with Chapter F, ksi (MPa)
Use the section modulus, S, for the specific location in the cross section and consider the sign of the stress.

w = subscript relating symbol to major principal axis bending

z = subscript relating symbol to minor principal axis bending

User Note: The subscripts w and z refer to the principal axes of the unsymmetric cross section. For doubly symmetric cross sections, these can be replaced by the x and y subscripts.

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 applicable. 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 SUBJECT TO TORSION AND COMBINED TORSION, FLEXURE, SHEAR, AND/OR AXIAL FORCE

1. Round and Rectangular HSS Subject to Torsion

The design torsional strength, $\phi_T T_n$, and the allowable torsional strength, T_n/Ω_T, for round and rectangular HSS according to the limit states of torsional yielding and torsional buckling shall be determined as follows:

$$T_n = F_{cr} C \quad \text{(H3-1)}$$

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

where

C = HSS torsional constant, in.^{3} (mm^3)

The critical stress, F_{cr}, shall be determined as follows:

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

(1) $$\frac{1.23E}{\sqrt{\frac{L}{D}}\left(\frac{D}{t}\right)^{\frac{5}{4}}} \quad \text{(H3-2a)}$$

and

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

but shall not exceed $0.6F_y$,

where

D = outside diameter, in. (mm)
L = length of member, in. (mm)
t = design wall thickness defined in Section B4.2, in. (mm)

(b) For rectangular HSS

(1) When $h/t \le 2.45\sqrt{E/F_y}$

$$F_{cr} = 0.6F_y \quad \text{(H3-3)}$$

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

$$F_{cr} = \frac{0.6F_y\left(2.45\sqrt{E/F_y}\right)}{\left(\frac{h}{t}\right)} \quad \text{(H3-4)}$$

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

$$F_{cr} = \frac{0.458\pi^2 E}{\left(\frac{h}{t}\right)^2} \quad \text{(H3-5)}$$

where

h = flat width of longer side, as defined in Section B4.1b(d), in. (mm)

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

For 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% of the available torsional strength, T_c, the interaction of torsion, shear, flexure and/or axial force for HSS may be determined by Section H1 and the torsional effects may be neglected. When T_r exceeds 20% of T_c, the interaction of torsion, shear, flexure and/or axial force shall be limited, at the point of consideration, by

$$\left(\frac{P_r}{P_c}+\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right)+\left(\frac{V_r}{V_c}+\frac{T_r}{T_c}\right)^2 \le 1.0 \qquad \text{(H3-6)}$$

where

P_r = required axial strength, determined in accordance with Chapter C, using LRFD or ASD load combinations, kips (N)

P_c = available tensile or compressive strength, ϕP_n or P_n/Ω, determined in accordance with Chapter D or E, kips (N)

M_r = required flexural strength, determined in accordance with Chapter C, using LRFD or ASD load combinations, kip-in. (N-mm)

M_c = available flexural strength, ϕM_n or M_n/Ω, determined in accordance with Chapter F, kip-in. (N-mm)

V_r = required shear strength, determined in accordance with Chapter C, using LRFD or ASD load combinations, kips (N)

V_c = available shear strength, ϕV_n or V_n/Ω, determined in accordance with Chapter G, kips (N)

T_r = required torsional strength, determined in accordance with Chapter C, using LRFD or ASD load combinations, kip-in. (N-mm)

T_c = available torsional strength, ϕT_n or T_n/Ω, determined in accordance with Section H3.1, kip-in. (N-mm)

3. Non-HSS Members Subject to Torsion and Combined Stress

The available torsional strength 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:

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

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

$$F_n = F_y \qquad \text{(H3-7)}$$

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

$$F_n = 0.6F_y \qquad \text{(H3-8)}$$

(c) For the limit state of buckling

$$F_n = F_{cr} \qquad \text{(H3-9)}$$

where

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

H4. RUPTURE OF FLANGES WITH HOLES AND SUBJECTED TO TENSION

At locations of bolt holes in flanges subjected to tension under combined axial force and major axis flexure, flange tensile rupture strength shall be limited by Equation H4-1. Each flange subjected to tension due to axial force and flexure shall be checked separately.

$$\frac{P_r}{P_c}+\frac{M_{rx}}{M_{cx}}\leq 1.0 \qquad \text{(H4-1)}$$

where

P_r = required axial strength of the member at the location of the bolt holes, determined in accordance with Chapter C, using LRFD or ASD load combinations, positive in tension and negative in compression, kips (N)

P_c = available axial strength for the limit state of tensile rupture of the net section at the location of bolt holes, ϕP_n or P_n/Ω, determined in accordance with Section D2(b), kips (N)

M_{rx} = required flexural strength at the location of the bolt holes, determined in accordance with Chapter C, using LRFD or ASD load combinations, positive for tension in the flange under consideration and negative for compression, kip-in. (N-mm)

M_{cx} = available flexural strength about x-axis for the limit state of tensile rupture of the flange, ϕM_n or M_n/Ω, determined according to Section F13.1. When the limit state of tensile rupture in flexure does not apply, use the plastic moment, M_p, determined with bolt holes not taken into consideration, kip-in. (N-mm)

CHAPTER I

DESIGN OF COMPOSITE MEMBERS

This chapter addresses composite members 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 steel headed stud anchors, as well as encased and filled beams, constructed with or without temporary shores, are included. This chapter also addresses concrete filled composite plate shear walls composed of structural steel plates, ties, steel anchors, and structural concrete acting together.

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.

1. Concrete and Steel Reinforcement

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. Additionally, the provisions in the *Building Code Requirements for Structural Concrete* (ACI 318) and the *Metric Building Code Requirements for Structural Concrete* (ACI 318M), subsequently referred to in Chapter I collectively as ACI 318, shall apply with the following exceptions and limitations:

(a) Concrete and steel reinforcement material limitations shall be as specified in Section I1.3.

(b) Longitudinal and transverse reinforcement requirements shall be as specified in Sections I2 and I3 in addition to those specified in ACI 318.

Concrete and steel reinforcement components designed in accordance with ACI 318 shall be based on a level of loading corresponding to LRFD load combinations.

User Note: It is the intent of this Specification that the concrete and reinforcing steel portions of composite concrete members are detailed utilizing the provisions of ACI 318 as modified by this Specification. All requirements specific to composite members are covered in this Specification.

Note that the design basis for ACI 318 is strength design. Designers using ASD for steel must be conscious of the different load factors.

2. Nominal Strength of Composite Sections

The nominal strength of composite sections shall be determined in accordance with either the plastic stress distribution method, the strain compatibility method, the elastic stress distribution method, or the effective stress-strain method, as defined in this section.

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

Local buckling effects shall be evaluated for filled composite members, as defined in Section I1.4. Local buckling effects need not be evaluated for encased composite columns or composite plate shear walls.

2a. 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 due to axial force and/or flexure have reached a stress of $0.85f_c'$, where f_c' is the specified compressive strength of concrete, ksi (MPa). For round HSS filled with concrete, a stress of $0.95f_c'$ is permitted to be used for concrete components in compression due to axial force and/or flexure to account for the effects of concrete confinement.

2b. 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.

User Note: The strain compatibility method can 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 members subjected to axial load, flexure or both are given in AISC Design Guide 6, *Load and Resistance Factor Design of W-Shapes Encased in Concrete*, and ACI 318.

2c. Elastic Stress Distribution Method

For the elastic stress distribution method, the nominal strength shall be determined from the superposition of elastic stresses for the limit state of yielding or concrete crushing.

2d. Effective Stress-Strain Method

For the effective stress-strain method, the nominal strength shall be computed assuming strain compatibility, and effective stress-strain relationships for steel and concrete components accounting for the effects of local buckling, yielding, interaction and concrete confinement.

3. Material Limitations

For concrete, structural steel, and steel reinforcing bars in composite systems, the following limitations shall be met unless Appendix X is used:

(a) For the determination of the available strength, concrete shall have a specified compressive strength, f_c', of not less than 3 ksi (21 MPa) nor more than 10 ksi (69 MPa) for normal weight concrete and not less than 3 ksi (21 MPa) nor more than 6 ksi (41 MPa) for lightweight concrete.

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(b) The specified minimum yield stress of structural steel used in calculating the strength of composite members shall not exceed 75 ksi (525 MPa).

(c) The specified minimum yield stress of reinforcing bars used in calculating the strength of composite members shall not exceed 80 ksi (550 MPa).

The design of filled composite members constructed from high-strength materials shall be in accordance with Appendix X.

4. Classification of Filled Composite Sections for Local Buckling

For compression, filled composite sections are classified as compact, noncompact or slender. For a section to qualify as compact, the maximum width-to-thickness ratio of its compression steel elements shall not exceed the limiting width-to-thickness ratio, λ_p, from Table I1.1a. If the maximum width-to-thickness ratio of one or more steel compression elements exceeds λ_p, but does not exceed λ_r from Table I1.1a, the filled composite section is noncompact. If the maximum width-to-thickness ratio of any compression steel element exceeds λ_r, the section is slender. The maximum permitted width-to-thickness ratio shall be as specified in the table.

For flexure, filled composite sections are classified as compact, noncompact or slender. For a section to qualify as compact, the maximum width-to-thickness ratio of its compression steel elements shall not exceed the limiting width-to-thickness ratio, λ_p, from Table I1.1b. If the maximum width-to-thickness ratio of one or more steel compression elements exceeds λ_p, but does not exceed λ_r from Table I1.1b, the section is noncompact. If the width-to-thickness ratio of any steel element exceeds λ_r, the section is slender. The maximum permitted width-to-thickness ratio shall be as specified in the table.

Refer to Section B4.1b for definitions of width, *b* and *D*, and thickness, *t*, for rectangular and round HSS sections and box sections of uniform thickness.

User Note: All current ASTM A1085 and ASTM A500 Grade C square HSS sections are compact according to the limits of Table I1.1a and Table I1.1b, except HSS7×7×1/8, HSS8×8×1/8, HSS10x10x3/16 and HSS12×12×3/16, which are noncompact for both axial compression and flexure, and HSS9x9x1/8, which is slender for both axial compression and flexure.

All current ASTM A500 Grade C round HSS sections are compact according to the limits of Table I1.1a and Table I1.1b for both axial compression and flexure, with the exception of HSS6.625x0.125, HSS7.000x0.125, HSS10.000x0.188, HSS14.000x0.250, HSS16.000×0.250, and HSS20.000x0.375, which are noncompact for flexure.

TABLE I1.1a
Limiting Width-to-Thickness Ratios for Compression Steel Elements in Composite Members Subject to Axial Compression for Use with Section I2.2

Description of Element	Width-to-Thickness Ratio	λ_p Compact/ Noncompact	λ_r Noncompact/ Slender	Maximum Permitted
Walls of Rectangular HSS and Box Sections of Uniform Thickness	b/t	$2.26\sqrt{\frac{E}{F_y}}$	$3.00\sqrt{\frac{E}{F_y}}$	$5.00\sqrt{\frac{E}{F_y}}$
Round HSS	D/t	$\frac{0.15E}{F_y}$	$\frac{0.19E}{F_y}$	$\frac{0.31E}{F_y}$

TABLE I1.1b
Limiting Width-to-Thickness Ratios for Compression Steel Elements in Composite Members Subject to Flexure for Use with Section I3.4

Description of Element	Width-to-Thickness Ratio	λ_p Compact/ Noncompact	λ_r Noncompact/ Slender	Maximum Permitted
Flanges of Rectangular HSS and Box Sections of Uniform Thickness	b/t	$2.26\sqrt{\frac{E}{F_y}}$	$3.00\sqrt{\frac{E}{F_y}}$	$5.00\sqrt{\frac{E}{F_y}}$
Webs of Rectangular HSS and Box Sections of Uniform Thickness	h/t	$3.00\sqrt{\frac{E}{F_y}}$	$5.70\sqrt{\frac{E}{F_y}}$	$5.70\sqrt{\frac{E}{F_y}}$
Round HSS	D/t	$\frac{0.09E}{F_y}$	$\frac{0.31E}{F_y}$	$\frac{0.31E}{F_y}$

5. Stiffness for Calculation of Required Strengths

For the direct analysis method of design, the required strengths of encased composite members, filled composite members, and composite plate shear walls shall be determined using the provisions of Section C2 and the following requirements:

(1) The nominal flexural stiffness of encased and filled composite members subject to net compression shall be taken as the effective stiffness of the composite section, EI_{eff}, as defined in Section I2.

(2) The nominal axial stiffness of encased and filled composite members subject to net compression shall be taken as the summation of the elastic axial stiffnesses of each component.

(3) Stiffness of encased and filled composite members subject to net tension shall be taken as the stiffness of the bare steel members in accordance with Chapter C.

(4) The stiffness reduction parameter, τ_b, shall be taken as 0.8 for encased and filled composite.

User Note: Taken together, the stiffness reduction factors require the use of $0.64EI_{eff}$ for the flexural stiffness and 0.8 times the nominal axial stiffness of encased composite members and filled composite members subject to net compression in the analysis.

Stiffness values appropriate for the calculation of deflections and for use with the effective length method are discussed in the Commentary.

(5) The flexural, axial, and shear stiffnesses of composite plate shear walls shall be calculated as follows:

$$(EI)_{eff} = E_s I_s + 0.35 E_c I_c \tag{I1-1}$$

$$(EA)_{eff} = E_s A_s + 0.45 E_c A_c \tag{I1-2}$$

$$(GA)_{eff} = G_s A_{sw} + G_c A_c \tag{I1-3}$$

where

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

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

A_{sw} = area of steel plates in the direction of in-plane shear, 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)

E_s = modulus of elasticity of steel
= 29,000 ksi (200,000 MPa)

G_s = shear modulus of steel
= 11,150 ksi (76,880 MPa)

G_c = shear modulus of concrete
= $0.4\,E_c$

I_c = moment of inertia of the concrete section about the elastic neutral axis of the composite section, in.^{4} (mm^4)

I_s = moment of inertia of steel shape about the elastic neutral axis of the composite section, in.^{4} (mm^4)

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

6. Requirements for Composite Plate Shear Walls

The opposing steel plates shall be connected to each other using *ties* consisting of bars, structural shapes, or built-up members. For filled composite plate shear walls, the steel plates shall be anchored to the concrete using ties or a combination of ties and steel anchors.

6a. Slenderness Requirement

The slenderness ratio of the plates, b/t, shall be limited as follows:

$$\frac{b}{t} \le 1.2\sqrt{\frac{E}{F_y}} \tag{I1-4}$$

where
b = largest clear distance between rows of steel anchors or ties, in. (mm)
t = plate thickness, in. (mm)

6b. Tie Bar Requirement

Tie bars shall have spacing no greater than 1.0 times the wall thickness, t_{sc} The tie bar spacing to plate thickness ratio, S/t, shall be limited as follows:

$$\frac{S}{t} \le 1.0\sqrt{\frac{E_s}{2\alpha + 1}} \tag{I1-5}$$

$$\alpha = 1.7\left[\frac{t_{sc}}{t} - 2\right]\left[\frac{t}{d_{tie}}\right]^4 \tag{I1-6}$$

where
S = largest clear spacing of the ties, in. (mm)
t = plate thickness, in. (mm)
t_{sc} = thickness of composite plate shear wall, in. (mm)
d_{tie} = effective diameter of the tie bar, in. (mm)

I2. AXIAL FORCE

This section applies to encased composite members, filled composite members, and composite plate shear walls subject to axial force.

1. Encased Composite Members

1a. Limitations

For encased composite members, the following limitations shall be met:

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

(b) Concrete encasement of the steel core shall be reinforced with continuous longitudinal bars and lateral ties or spirals.

Detailing of longitudinal reinforcing, including bar spacing and concrete cover requirements, shall conform to ACI 318.

Transverse reinforcement shall consist of a minimum of either a No. 3 (10 mm) bar spaced at a maximum of 12 in. (300 mm) on center, or a No. 4 (13 mm) bar or larger spaced at a maximum of 16 in. (400 mm) on center shall be used. Deformed wire or welded wire reinforcement of equivalent area are permitted.

Maximum spacing of lateral ties shall not exceed 0.5 times the least column dimension. Refer to ACI 318 for concrete cover requirements.

(c) 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_g = gross area of composite member, in.^{2} (mm^2)

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

(d) The maximum reinforcement ratio for continuous longitudinal reinforcing, ρ_{sr}, shall be based on ACI 318 with the gross area of concrete A_g assumed in the calculations.

User Note: Refer to ACI 318 for additional longitudinal steel, lateral tie, and spiral reinforcing provisions. Refer to Section I4 for shear requirements.

1b. Compressive Strength

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

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

(a) When $\dfrac{P_{no}}{P_e} \le 2.25$

$$P_n = P_{no}\left(0.658^{\frac{P_{no}}{P_e}}\right) \tag{I2-2}$$

(b) When $\dfrac{P_{no}}{P_e} > 2.25$

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

where

$P_{no} = F_y A_s + F_{ysr} A_{sr} + 0.85 f_c' A_c$ (I2-4)

P_e = elastic critical buckling load determined in accordance with Chapter C or Appendix 7, kips (N)

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

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

A_s = cross-sectional area of steel section, 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)

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

$= E_s I_s + E_s I_{sr} + C_1 E_c I_c$ (I2-6)

C_1 = coefficient for calculation of effective rigidity of an encased composite compression member

$= 0.25 + 3\left(\dfrac{A_s + A_{sr}}{A_g}\right) \le 0.7$ (I2-7)

E_s = modulus of elasticity of steel

= 29,000 ksi (200 000 MPa)
F_y = specified minimum yield stress of steel section, ksi (MPa)
F_{ysr} = specified minimum yield stress of reinforcing steel, ksi (MPa)
I_c = moment of inertia of the concrete section about the elastic neutral axis of the composite section, in.^{4} (mm^4)
I_s = moment of inertia of steel shape about the elastic neutral axis of the composite section, in.^{4} (mm^4)
I_{sr} = moment of inertia of reinforcing bars about the elastic neutral axis of the composite section, in.^{4} (mm^4)
K = effective length factor
L = laterally unbraced length of the member, in. (mm)
L_c = KL = effective length of the member, in. (mm)
f_c' = specified compressive strength of concrete, ksi (MPa)
w_c = weight of concrete per unit volume ($90 \le w_c \le 155$ lb/ft^3 or $1500 \le w_c \le 2500$ kg/m^3)

The available compressive strength need not be less than that specified for the bare steel member, as required by Chapter E.

1c. Tensile Strength

The available tensile strength of axially loaded encased composite members shall be determined for the limit state of yielding as:

$$P_n = F_y A_s + F_{ysr} A_{sr} \quad \text{(I2-8)}$$

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

1d. Load Transfer

Load transfer requirements for encased composite members shall be determined in accordance with Section I6.

1e. Detailing Requirements

For encased composite members, the following detailing requirements shall be met:

(a) Clear spacing between the steel core and longitudinal reinforcing shall be a minimum of 1.5 reinforcing bar diameters, but not less than 1.5 in. (38 mm).

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

User Note: Refer to ACI 318 for additional longitudinal steel, lateral tie, and spiral reinforcing provisions. Refer to Section I4 for shear requirements.

2. Filled Composite Members

2a. Limitations

For filled composite members, the following limitations shall be met:

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

(b) Filled composite members shall be classified for local buckling according to Section I1.4.

(c) Minimum longitudinal reinforcement is not required. If longitudinal reinforcement is provided, internal transverse reinforcement is not required for strength; however, minimum internal transverse reinforcement shall be provided for constructability. A minimum of either a No. 3 (10 mm) bar spaced at a maximum of 12 in. (300 mm) on center, or a No. 4 (13 mm) bar or larger spaced at a maximum of 16 in. (400 mm) on center shall be used. Deformed wire or welded wire reinforcement of equivalent area are permitted.

(d) If longitudinal reinforcing steel is provided for strength, the maximum reinforcement ratio shall be based on ACI 318 requirements for the gross area of concrete

User Note: Refer to ACI 318 for additional longitudinal steel, lateral tie, and spiral reinforcing provisions. Refer to Section I4 and Section I4 Commentary for shear in concrete filled members.

2b. Compressive Strength

The available compressive strength of axially loaded doubly symmetric filled composite members shall be determined for the limit state of flexural buckling in accordance with Section I2.1b with the following modifications:

(a) For compact sections

$$P_{no} = P_p \tag{I2-9a}$$

where

$$P_p = F_y A_s + C_2 f_c' \left(A_c + A_{sr} \frac{E_s}{E_c} \right) \tag{I2-9b}$$

C_2 = 0.85 for rectangular sections and 0.95 for round sections

(b) For noncompact sections

$$P_{no} = P_p - \frac{P_p - P_y}{\left(\lambda_r - \lambda_p\right)^2}\left(\lambda - \lambda_p\right)^2 \tag{I2-9c}$$

where

λ, λ_p and λ_r are width-to-thickness ratios determined from Table I1.1a

P_p is determined from Equation I2-9b

$$P_y = F_y A_s + 0.7 f_c' \left(A_c + A_{sr} \frac{E_s}{E_c} \right) \tag{I2-9d}$$

(c) For slender sections

$$P_{no} = F_{cr} A_s + 0.7 f'_c \left(A_c + A_{sr} \frac{E_s}{E_c} \right) \quad \text{(I2-9e)}$$

where

(1) For rectangular filled sections

$$F_{cr} = \frac{9E_s}{\left(\dfrac{b}{t}\right)^2} \quad \text{(I2-10)}$$

(2) For round filled sections

$$F_{cr} = \frac{0.72F_y}{\left[\left(\dfrac{D}{t}\right)\dfrac{F_y}{E_s}\right]^{0.2}} \quad \text{(I2-11)}$$

The effective stiffness of the composite section, EI_{eff}, for all sections shall be:

$$EI_{eff} = E_s I_s + E_s I_{sr} + C_3 E_c I_c \quad \text{(I2-12)}$$

where

C_3 = coefficient for calculation of effective rigidity of filled composite compression member

$$= 0.45 + 3\left(\frac{A_s + A_{sr}}{A_g}\right) \le 0.9 \quad \text{(I2-13)}$$

The available compressive strength need not be less than specified for the bare steel member, as required by Chapter E.

2c. Tensile Strength

The available tensile strength of axially loaded filled composite members shall be determined for the limit state of yielding as:

$$P_n = A_s F_y + A_{sr} F_{ysr} \quad \text{(I2-14)}$$

$\phi_t = 0.90$ (LRFD) $\qquad \Omega_t = 1.67$ (ASD)

2d. Load Transfer

Load transfer requirements for filled composite members shall be determined in accordance with Section I6.

2e. Detailing Requirements

Clear spacing between the inside steel perimeter and longitudinal reinforcing where provided shall be a minimum of 1.5 reinforcing bar diameters, but not less than 1.5 in. (38 mm).

3. Composite Plate Shear Walls

3a. Limitations

For composite plate shear walls, the following limitations shall be met:

(a) The steel plates shall comprise at least 1% but no more than 10% of the total composite cross-section area.

(b) The steel plates shall satisfy the slenderness requirements of Section I1.6.

(c) Walls without flange (closure) plates or boundary elements are not permitted.

(d) The height-to-length ratio of the wall shall be greater than or equal to 3.

3b. Compressive Strength

The available compressive strength of axially loaded composite plate shear walls shall be determined for the limit state of flexural buckling in accordance with Section I2.1b. The value of flexural stiffness from Section I1.5 shall be used along with the section axial load capacity, P_{no}, determined as follows:

$$P_{no} = F_y A_s + 0.85 f_c' A_c \qquad \text{(I2-15)}$$

$\phi_c = 0.90$ (LRFD) $\qquad$ $\Omega_t = 1.67$ (ASD)

3c. Tensile Strength

The available tensile strength of axially loaded composite plate shear walls shall be determined for the limit state of yielding as:

$$P_n = A_s F_y \qquad \text{(I2-16)}$$

$\phi_t = 0.90$ (LRFD) $\qquad$ $\Omega_t = 1.67$ (ASD)

I3. FLEXURE

This section applies to three types of composite members subject to flexure: composite beams with steel anchors consisting of steel headed stud anchors or steel channel anchors, concrete encased members, and concrete filled members.

1. General

1a. Effective Width

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

(a) one-eighth of the beam span, center-to-center of supports;
(b) one-half the distance to the centerline of the adjacent beam; or
(c) the distance to the edge of the slab.

1b. Strength During Construction

When temporary shores are not used during construction, the steel section alone shall have sufficient strength to support all loads applied prior to the concrete attaining 75% of its specified strength, f_c'. The available flexural strength of the steel section shall be determined in accordance with Chapter F.

2. Composite Beams with Steel Headed Stud or Steel Channel Anchors

2a. Positive Flexural Strength

The design positive flexural strength, $\phi_b M_n$, and 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) When $h/t_w \le 3.76\sqrt{E/F_y}$

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 $F_y \le 70$ ksi (485 MPa).

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

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 available negative flexural strength 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 the following limitations are met:

(a) The steel beam is compact and is adequately braced in accordance with Chapter F.

(b) Steel headed stud or steel channel anchors connect the slab to the steel beam in the negative moment region.

(c) The slab reinforcement parallel to the steel beam, within the effective width of the slab, is developed.

User Note: To check compactness of a composite beam in negative flexure, Case 10 in Table B4.1 is appropriate to use for flanges, and Case 16 of Table B4.1 is appropriate to use for webs.

2c. 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 Sections I3.2a and I3.2b, with the following requirements:

(a) The nominal rib height shall not be 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 steel headed stud anchors welded either through the deck or directly to the steel cross section. Steel headed stud anchors, 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 specified concrete cover above the top of the steel headed stud anchors.

(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 steel headed stud anchors, a combination of steel headed stud anchors and arc spot (puddle) welds, or other devices specified by the contract documents.

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 is permitted to be included in determining composite section properties and in calculating A_c.

Formed steel deck ribs over supporting beams are permitted to 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 steel headed stud anchor in the transverse row plus four stud diameters for each additional steel headed stud anchor.

2d. Load Transfer Between Steel Beam and Concrete Slab

1. Load Transfer for Positive Flexural Strength

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

(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) Shear strength of steel headed stud or steel channel anchors

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

where

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

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

ΣQ_n = sum of nominal shear strengths of steel headed stud or steel channel anchors between the point of maximum positive moment and the point of zero moment, kips (N)

The effect of ductility (slip capacity) of the shear connection at the interface of the concrete slab and the steel beam shall be considered.

2. Load Transfer for Negative Flexural Strength

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 between the point of maximum negative moment and the point of zero moment shall be determined as the lower value in accordance with the following limit states:

(a) For the limit state of tensile yielding of the slab reinforcement

$$V' = F_{ysr} A_{sr} \tag{I3-2a}$$

where

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

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

(b) For the limit state of shear strength of steel headed stud or steel channel anchors

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

3. Encased Composite Members

3a. Limitations

For encased composite members, the following limitations shall be met:

(a) The available flexural strength of concrete-encased members shall be determined as follows:

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

The nominal flexural strength, M_n, shall be determined using one of the following methods:

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

(2) The plastic stress distribution on the steel section alone, for the limit state of yielding (plastic moment) on the steel section.

(3) The plastic stress distribution on the composite section or the strain-compatibility method, for the limit state of yielding (plastic moment) on the composite section. For concrete-encased members, steel anchors shall be provided.

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

(c) Concrete encasement of the steel core shall be reinforced with continuous longitudinal bars and transverse reinforcement (stirrups).

Detailing of longitudinal reinforcing, including bar spacing and concrete cover requirements, shall conform to ACI 318.

Transverse reinforcement for constructability shall consist of a minimum of either a No. 3 (10 mm) bar spaced at a maximum of 12 in. (300 mm) on center, or a No. 4 (13 mm) bar or larger spaced at a maximum of 16 in. (400 mm) on center shall be used. Deformed wire or welded wire reinforcement of equivalent area are permitted.

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

$$\rho_{sr} = \frac{A_{sr}}{A_g} \qquad \text{(I3-3)}$$

where

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

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

(e) The composite member, including the area of the steel section and reinforcing steel, shall be tension controlled as defined in ACI 318.

User Note: The effect of this limitation is to restrict the reinforcement ratio to mitigate brittle fracture behavior in case of an overload. Refer to ACI 318 for additional longitudinal steel, lateral tie and spiral reinforcing provisions. Refer to Section I4 for shear requirements.

3b. Detailing Requirements

Clear spacing between the steel core and longitudinal reinforcing shall be a minimum of 1.5 reinforcing bar diameters, but not less than 1.5 in. (38 mm).

4. Filled Composite Members

4a. Limitations

(a) Filled composite sections shall be classified for local buckling according to Section I1.4.

(b) The total cross sectional area of the steel core shall comprise at least 1% of the total composite cross section

c) Minimum longitudinal reinforcement is not required.

Where provided, the minimum reinforcement ratio for continuous longitudinal reinforcing, ρ_{sr}, shall be 0.004, where ρ_{sr} is given by:

$$\rho_{sr} = \frac{A_{sr}}{A_g} \tag{I3-4}$$

If longitudinal reinforcement is provided, internal transverse reinforcement is not required for strength; however, minimum internal transverse reinforcement shall be provided for constructability. A minimum of either a No. 3 (10 mm) bar spaced at a maximum of 12 in. (300 mm) on center, or a No. 4 (13 mm) bar or larger spaced at a maximum of 16 in. (400 mm) on center shall be used. Deformed wire or welded wire reinforcement of equivalent area are permitted

(d) The composite member, including the area of the steel section and reinforcing steel, shall be tension controlled as defined in ACI 318.

User Note: The effect of this limitation is to restrict the reinforcement ratio to mitigate brittle fracture behavior in case of an overload. Refer to ACI 318 for additional longitudinal steel, lateral tie, and spiral reinforcing provisions. Refer to Section I4 for shear requirements.

4b. Flexural Strength

The available flexural strength of filled composite members shall be determined as follows:

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

The nominal flexural strength, M_n, shall be determined as follows:

(a) For compact sections

$$M_n = M_p \tag{I3-3a}$$

where

M_p = moment corresponding to plastic stress distribution over the composite cross section, kip-in. (N-mm)

(b) For noncompact sections

$$M_n = M_p - \left(M_p - M_y\right)\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right) \qquad \text{(I3-3b)}$$

where

λ, λ_p and λ_r are width-to-thickness ratios determined from Table I1.1b.

M_y = yield moment corresponding to yielding of the tension flange and first yield of the compression flange, kip-in. (N-mm). The capacity at first yield shall be calculated assuming a linear elastic stress distribution with the maximum concrete compressive stress limited to $0.7\,f_c'$ and the maximum steel stress limited to F_y.

(c) For slender sections, M_n, shall be determined as the first yield moment. The compression flange stress shall be limited to the local buckling stress, F_{cr}, determined using Equation I2-10 or I2-11. The concrete stress distribution shall be linear elastic with the maximum compressive stress limited to $0.70f_c'$.

4c. Detailing Requirements

Clear spacing between the inside steel perimeter and longitudinal reinforcing where provided shall be a minimum of 1.5 reinforcing bar diameters, but not less than 1.5 in. (38 mm).

5. Composite Plate Shear Walls

The available flexural strength of filled composite plate shear walls shall be determined in accordance with section I1.2a as the moment, M_p, corresponding to plastic stress distribution over the composite cross section.

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

I4. SHEAR

1. Encased Composite Members

The design shear strength, $\phi_v V_n$, and allowable shear strength, V_n/Ω_v, shall be determined based on one of the following:

(a) The available shear strength of the steel section alone as specified in Chapter G

(b) The available shear strength of the reinforced concrete portion (concrete plus steel reinforcement) alone as defined by ACI 318 with

$\phi_v = 0.75$ (LRFD) $\qquad$ $\Omega_v = 2.00$ (ASD)

(c) The nominal shear strength of the steel section, as defined in Chapter G, plus the nominal strength of the reinforcing steel, as defined by ACI 318, with a combined resistance or safety factor of

$\phi_v = 0.75$ (LRFD) $\Omega_v = 2.00$ (ASD)

2. Filled Composite Members

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

$\phi_v = 0.90$ (LRFD) $\Omega_v = 1.67$ (ASD)

The nominal shear strength, V_n, shall account for the contributions of the steel section and concrete infill as follows:

$$V_n = 0.6A_v F_y + 0.06K_c A_c \sqrt{f_c'} \tag{I4-1}$$

where

A_v = Shear area of the steel portion of a composite member. The shear area for a circular section is equal to $2A_s/\pi$, and for a rectangular section is equal to the sum of the area of webs in the direction of in-plane shear, in.^{2} (mm^2)

A_c = Area of concrete in filled composite member, in.^{2} (mm^2)

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

K_c = 1 for members with shear span-to-depth ($M_u/V_u d$) greater than or equal to 0.7, where M_u and V_u are equal to the maximum moment and shear demands, respectively, along the member length, and d is equal to the member depth in the direction of bending

K_c = 10 for members with rectangular compact cross sections and $M_u/V_u d$ less than 0.5

K_c = 9 for members with circular compact cross sections and $M_u/V_u d$ less than 0.5

K_c = 1 for members having other than compact cross sections

f'_c = concrete strength in ksi

Linear interpolation between the above K_c values shall be used for members with compact cross sections and $M_u/V_u d$ between 0.5 and 0.7.

3. Composite Beams with Formed Steel Deck

The available shear strength of composite beams with steel headed stud or steel channel anchors shall be determined based upon the properties of the steel section alone in accordance with Chapter G.

4. Composite Plate Shear Walls

The design in-plane shear strength, $\phi_v V_n$, and allowable shear strength, V_n/Ω_v, of composite plate shear walls shall be determined as follows:

$\phi_v = 0.90$ (LRFD) $\Omega_v = 1.67$ (ASD)

The nominal shear strength, V_n, shall account for the contributions of the steel section and concrete infill as follows:

$$V_n = \frac{K_s + K_{sc}}{\sqrt{3K_s^2 + K_{sc}^2}} A_{sw} F_y \tag{I4-2}$$

where

A_{sw} = area of steel plates in the direction of in-plane shear

G = shear modulus of steel, ksi (MPa)

$$K_s = GA_{sw} \tag{I4-3}$$

$$K_{sc} = \frac{0.7\left(E_c A_c\right)\left(E_s A_{sw}\right)}{4E_s A_{sw} + E_c A_c} \tag{I4-4}$$

I5. COMBINED FLEXURE AND AXIAL FORCE

The interaction between flexure and axial forces in composite members shall account for stability as required by Chapter C. The available compressive strength and the available flexural strength shall be determined as defined in Sections I2 and I3, respectively. 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 in accordance with Section I2.

(a) For encased composite members and for filled composite members with compact sections, the interaction between axial force and flexure shall be based on the interaction equations of Section H1.1 or one of the methods defined in Section I1.2.

(b) For filled composite members with noncompact or slender sections, the interaction between axial force and flexure shall be based either on the interaction equations of Section H1.1, the method defined in Section I1.2d, or Equations I5-1a and b.

(1) When $\dfrac{P_r}{P_c} \geq c_p$

$$\frac{P_r}{P_c} + \frac{1 - c_p}{c_m}\left(\frac{M_r}{M_c}\right) \leq 1.0 \tag{I5-1a}$$

(2) When $\dfrac{P_r}{P_c} < c_p$

$$\left(\frac{1 - c_m}{c_p}\right)\left(\frac{P_r}{P_c}\right) + \frac{M_r}{M_c} \leq 1.0 \tag{I5-1b}$$

Table I5.1
Coefficients c_p and c_m for Use with Equations I5-1a and I5-1b

Filled Composite Member Type	c_p	c_m	
		when $c_{sr} \geq 0.5$	when $c_{sr} < 0.5$
Rectangular	$c_p = \dfrac{0.17}{c_{sr}^{\;0.4}}$	$c_m = \dfrac{1.06}{c_{sr}^{\;0.11}} \geq 1.0$	$c_m = \dfrac{0.90}{c_{sr}^{\;0.36}} \leq 1.67$

Round HSS	$c_p = \dfrac{0.27}{c_{sr}^{\ 0.4}}$	$c_m = \dfrac{1.10}{c_{sr}^{\ 0.08}} \geq 1.0$	$c_m = \dfrac{0.95}{c_{sr}^{\ 0.32}} \leq 1.67$

where

M_c = available flexural strength, determined in accordance with Section I3, kip-in. (N-mm)

M_r = required flexural strength, determined in accordance with Section I1.5, using LRFD or ASD load combinations, kip-in. (N-mm)

P_c = available axial strength, determined in accordance with Section I2, kips (N)

P_r = required axial strength, determined in accordance with Section I1.5, using LRFD or ASD load combinations, kips (N)

For design according to Section B3.1 (LRFD):

M_c = $\phi_b M_n$ = design flexural strength determined in accordance with Section I3, kip-in. (N-mm)

M_r = required flexural strength, determined in accordance with Section I1.5, using LRFD load combinations, kip-in. (N-mm)

P_c = $\phi_c P_n$ = design axial strength, determined in accordance with Section I2, kips (N)

P_r = required axial strength, determined in accordance with Section I1.5, using LRFD load combinations, kips (N)

ϕ_c = resistance factor for compression = 0.75

ϕ_b = resistance factor for flexure = 0.90

For design according to Section B3.2 (ASD):

M_c = M_n/Ω_b = allowable flexural strength, determined in accordance with Section I3, kip-in. (N-mm)

M_r = required flexural strength, determined in accordance with Section I1.5, using ASD load combinations, kip-in. (N-mm)

P_c = P_n/Ω_c = allowable axial strength, determined in accordance with Section I2, kips (N)

P_r = required axial strength, determined in accordance with Section I1.5, using ASD load combinations, kips (N)

Ω_c = safety factor for compression = 2.00

Ω_b = safety factor for flexure = 1.67

c_m and c_p are determined from Table I5.1

$$c_{sr} = \frac{A_s F_y + A_{sr} F_{yr}}{A_c f_c'} \qquad \text{(I5-2)}$$

(c) For filled composite plate shear wall sections, the interaction between axial force and flexure shall be based on methods defined in Section I1.2a or I1.2d.

I6. LOAD TRANSFER

1. General Requirements

When external forces are applied to an axially loaded encased or filled composite member, the introduction of force to the member and the transfer of longitudinal shear within the member shall be assessed in accordance with the requirements for force allocation presented in this section.

The available strength of the applicable force transfer mechanisms as determined in accordance with Section I6.3 shall equal or exceed the required longitudinal shear force to be transferred, V_r', as determined in accordance with Section I6.2. Force transfer mechanisms shall be located within the load transfer region as determined in accordance with Section I6.4.

2. Force Allocation

Force allocation shall be determined based upon the distribution of external force in accordance with the following requirements.

User Note: Bearing strength provisions for externally applied forces are provided in Section J8. For filled composite members, the term $\sqrt{A_2/A_1}$ in Equation J8-2 may be taken equal to 2.0 due to confinement effects.

2a. External Force Applied to Steel Section

When the entire external force is applied directly to the steel section, the force required to be transferred to the concrete, V_r', shall be determined as:

$$V_r' = P_r\left(1 - F_y A_s / P_{no}\right) \quad \text{(I6-1)}$$

where

P_{no} = nominal axial compressive strength without consideration of length effects, determined by Equation I2-4 for encased composite members, and Equation I2-9a or Equation I2-9c, as applicable, for compact or noncompact filled composite members, kips (N)

P_r = required external force applied to the composite member, kips (N)

User Note: Equation I6-1 does not apply to slender filled composite members for which the external force is applied directly to the concrete fill in accordance with Section I6.2b, or concurrently to the steel and concrete, in accordance with Section I6.2c.

2b. External Force Applied to Concrete

When the entire external force is applied directly to the concrete encasement or concrete fill, the force required to be transferred to the steel, V_r', shall be determined as follows:

(a) For encased or filled composite members that are compact or noncompact

$$V_r' = P_r\left(F_y A_s / P_{no}\right) \quad \text{(I6-2a)}$$

(b) For slender filled composite members

$$V_r' = P_r\left(F_{cr} A_s / P_{no}\right) \quad \text{(I6-2b)}$$

where

F_{cr} = critical buckling stress for steel elements of filled composite members determined using Equation I2-10 or Equation I2-11, as applicable, ksi (MPa)

P_{no} = nominal axial compressive strength without consideration of length effects, determined by Equation I2-4 for encased composite members, and Equation I2-9a for filled composite members, kips (N)

2c. External Force Applied Concurrently to Steel and Concrete

When the external force is applied concurrently to the steel section and concrete encasement or concrete fill, V_r' shall be determined as the force required to establish equilibrium of the cross section.

User Note: The Commentary provides an acceptable method of determining the longitudinal shear force required for equilibrium of the cross section.

3. Force Transfer Mechanisms

The available strength of the force transfer mechanisms of direct bond interaction, shear connection, and direct bearing shall be determined in accordance with this section. Use of the force transfer mechanism providing the largest nominal strength is permitted. Force transfer mechanisms shall not be superimposed.

The force transfer mechanism of direct bond interaction shall not be used for encased composite members or for filled composite members where bond failure would result in uncontrolled slip.

3a. Direct Bearing

Where force is transferred in an encased or filled composite member by direct bearing from internal bearing mechanisms, the available bearing strength of the concrete for the limit state of concrete crushing shall be determined as:

$$R_n = 1.7 f_c' A_1 \tag{I6-3}$$

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

where

A_1 = loaded area of concrete, in.^{2} (mm^2)

User Note: An example of force transfer via an internal bearing mechanism is the use of internal steel plates within a filled composite member.

3b. Shear Connection

Where force is transferred in an encased or filled composite member by shear connection, the available shear strength of steel headed stud or steel channel anchors shall be determined as:

$$R_c = \Sigma Q_{cv} \tag{I6-4}$$

where

ΣQ_{cv} = sum of available shear strengths, $\phi_v Q_{nv}$ (LRFD) or Q_{nv}/Ω_v (ASD), as applicable, of steel headed stud or steel channel anchors, determined in accordance with Section I8.3a or Section I8.3d, respectively, placed within the load introduction length as defined in Section I6.4, kips (N)

3c. Direct Bond Interaction

Where force is transferred in a filled composite member by direct bond interaction, the available bond strength between the steel and concrete shall be determined as follows:

$$R_n = p_b L_{in} F_{in} \tag{I6-5}$$

$$\phi_d = 0.50 \text{ (LRFD)} \qquad \Omega_d = 3.00 \text{ (ASD)}$$

where

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

F_{in} = nominal bond stress, ksi (MPa)

= $12t/H^2 \le 0.1$, ksi ($2100t/H^2 \le 0.7$, MPa) for rectangular cross sections

= $30t/D^2 \le 0.2$, ksi ($5300t/D^2 \le 1.4$, MPa) for circular cross sections

H = maximum transverse dimension of rectangular steel member, in. (mm)

L_{in} = load introduction length, determined in accordance with Section I6.4, in. (mm)

R_n = nominal bond strength, kips (N)

p_b = perimeter of the steel-concrete bond interface within the composite cross section, in. (mm)

t = design wall thickness of HSS member as defined in Section B4.2, in. (mm)

4. Detailing Requirements

4a. Encased Composite Members

Force transfer mechanisms shall be distributed within the load introduction length, which shall not exceed a distance of two times the minimum transverse dimension of the encased composite member above and below the load transfer region. Anchors utilized to transfer longitudinal shear shall be placed on at least two faces of the steel shape in a generally symmetric configuration about the steel shape axes.

Steel anchor spacing, both within and outside of the load introduction length, shall conform to Section I8.3e.

4b. Filled Composite Members

Force transfer mechanisms shall be distributed within the load introduction length, which shall not exceed a distance of two times the minimum transverse dimension of a rectangular steel member or two times the diameter of a round steel member both above and below the load transfer region. For the specific case of load applied to the concrete of a filled composite member containing no internal reinforcement, the load introduction length shall extend beyond the load transfer region in only the direction of the applied force. Steel anchor spacing within the load introduction length shall conform to Section I8.3e.

I7. COMPOSITE DIAPHRAGMS AND COLLECTOR BEAMS

Composite slab diaphragms and collector beams shall be designed and detailed to transfer loads between the diaphragm, the diaphragm's boundary members and collector elements, and elements of the lateral force-resisting system.

User Note: Design guidelines for composite diaphragms and collector beams can be found in the Commentary.

I8. STEEL ANCHORS

1. General

The diameter of a steel headed stud anchor, d_{sa}, shall be 3/4 in. (19 mm) or less, except where anchors are utilized solely for shear transfer in solid slabs in which case 7/8-in.- (2 mm) and 1-in.- (25 mm) diameter anchors are permitted. Additionally, d_{sa} shall not be greater than 2.5 times the thickness of the base metal to which it is welded, unless it is welded to a flange directly over a web.

Section I8.2 applies to a composite flexural member where steel anchors are embedded in a solid concrete slab or in a slab cast on formed steel deck. Section I8.3 applies to all other cases.

2. Steel Anchors in Composite Beams

The length of steel headed stud anchors shall not be less than four stud diameters from the base of the steel headed stud anchor to the top of the stud head after installation.

2a. Strength of Steel Headed Stud Anchors

The nominal shear strength of one steel headed stud anchor embedded in a solid concrete slab or in a composite slab with decking shall be determined as follows:

$$Q_n = 0.5A_{sa}\sqrt{f_c' E_c} \leq R_g R_p A_{sa} F_u \quad \text{(I8-1)}$$

where

A_{sa} = cross-sectional area of steel headed stud anchor, 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 steel headed stud anchor, ksi (MPa)

R_g = 1.0 for:

(a) One steel headed stud anchor welded in a steel deck rib with the deck oriented perpendicular to the steel shape

(b) Any number of steel headed stud anchors welded in a row directly to the steel shape

(c) Any number of steel headed stud anchors 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 for:

(a) Two steel headed stud anchors welded in a steel deck rib with the deck oriented perpendicular to the steel shape

(b) One steel headed stud anchor 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 steel headed stud anchors welded in a steel deck rib with the deck oriented perpendicular to the steel shape

R_p = 0.75 for:

(a) Steel headed stud anchors welded directly to the steel shape

(b) Steel headed stud anchors welded in a composite slab with the deck oriented perpendicular to the beam and $e_{mid\text{-}ht}$ ≥ 2 in. (50 mm)

(c) Steel headed stud anchors 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 steel headed stud anchors 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 steel headed stud anchor shank to the steel deck web, measured at mid-height of the deck rib, and in the load bearing direction of the steel headed stud anchor (in other words, in the direction of maximum moment for a simply supported beam), in. (mm)

User Note: The table below presents values for R_q and R_p for several cases. Available strengths for steel headed stud anchors can be found in the AISC *Steel Construction Manual*.

Condition	R_g	R_p
No decking	1.0	0.75
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[a]	0.75
Decking oriented perpendicular to the steel shape Number of steel headed stud anchors occupying the same decking rib:		
1	1.0	0.6[b]
2	0.85	0.6[b]
3 or more	0.7	0.6[b]

h_r = nominal rib height, in. (mm)

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

[a] For a single steel headed stud anchor

[b] This value may be increased to 0.75 when $e_{mid\text{-}ht}$ ≥ 2 in. (50 mm).

2b. Strength of Steel Channel Anchors

The nominal shear strength of one hot-rolled channel anchor embedded in a solid concrete slab shall be determined as:

$$Q_n = 0.3(t_f + 0.5t_w)l_a\sqrt{f'_c E_c} \qquad \text{(I8-2)}$$

where

l_a = length of channel anchor, in. (mm)
t_f = thickness of flange of channel anchor, in. (mm)
t_w = thickness of channel anchor web, in. (mm)

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

2c. Required Number of Steel Anchors

The number of anchors 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 as determined in Sections I3.2d.1 and I3.2d.2 divided by the nominal shear strength of one steel anchor as determined from Section I8.2a or Section I8.2b. The number of steel anchors required 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.

2d. Detailing Requirements

Steel anchors in composite beams shall meet the following requirements:

(a) Steel anchors 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 specified otherwise on the contract documents.

(b) Steel anchors shall have at least 1 in. (25 mm) of lateral concrete cover in the direction perpendicular to the shear force, except for anchors installed in the ribs of formed steel decks.

(c) The minimum distance from the center of a steel anchor to a free edge in the direction of the shear force shall be 8 in. (200 mm) if normal weight concrete is used and 10 in. (250 mm) if lightweight concrete is used. The provisions of ACI 318 Chapter 17 are permitted to be used in lieu of these values.

(d) Minimum center-to-center spacing of steel headed stud anchors shall be four diameters in any direction. For composite beams that do not contain anchors located within formed steel deck oriented perpendicular to the beam span, an additional minimum spacing limit of six diameters along the longitudinal axis of the beam shall apply.

(e) The maximum center-to-center spacing of steel anchors shall not exceed eight times the total slab thickness or 36 in. (900 mm).

3. Steel Anchors in Composite Components

This section shall apply to the design of cast-in-place steel headed stud anchors and steel channel anchors in composite components.

The provisions of the applicable building code or ACI 318 Chapter 17 are permitted to be used in lieu of the provisions in this section.

User Note: The steel headed stud anchor strength provisions in this section are applicable to anchors located primarily in the load transfer (connection) region of composite columns and beam-columns, concrete-encased and filled composite beams, composite coupling beams, and composite walls, where the steel and concrete are working compositely within a member. They are not intended for hybrid construction where the steel and concrete are not working compositely, such as with embed plates.

Section I8.2 specifies the strength of steel anchors embedded in a solid concrete slab or in a concrete slab with formed steel deck in a composite beam.

Limit states for the steel shank of the anchor and for concrete breakout in shear are covered directly in this Section. Additionally, the spacing and dimensional limitations provided in these provisions preclude the limit states of concrete pryout for anchors loaded in shear and concrete breakout for anchors loaded in tension as defined by ACI 318 Chapter 17.

For normal weight concrete: Steel headed stud anchors subjected to shear only shall not be less than five stud diameters in length from the base of the steel headed stud to the top of the stud head after installation. Steel headed stud anchors subjected to tension or interaction of shear and tension shall not be less than eight stud diameters in length from the base of the stud to the top of the stud head after installation.

For lightweight concrete: Steel headed stud anchors subjected to shear only shall not be less than seven stud diameters in length from the base of the steel headed stud to the top of the stud head after installation. Steel headed stud anchors subjected to tension shall not be less than ten stud diameters in length from the base of the stud to the top of the stud head after installation. The nominal strength of steel headed stud anchors subjected to interaction of shear and tension for lightweight concrete shall be determined as stipulated by the applicable building code or ACI 318 Chapter 17.

Steel headed stud anchors subjected to tension or interaction of shear and tension shall have a diameter of the head greater than or equal to 1.6 times the diameter of the shank.

User Note: The following table presents values of minimum steel headed stud anchor *h/d* ratios for each condition covered in this Specification.

Loading Condition	**Normal Weight Concrete**	**Lightweight Concrete**
Shear	$h/d_{sa} \geq 5$	$h/d_{sa} \geq 7$
Tension	$h/d_{sa} \geq 8$	$h/d_{sa} \geq 10$
Shear and Tension	$h/d_{sa} \geq 8$	N/A[a]
h/d_{sa} = ratio of steel headed stud anchor shank length to the top of the stud head, to shank diameter.		

[a] Refer to ACI 318 Chapter 17 for the calculation of interaction effects of anchors embedded in lightweight concrete.

3a. Shear Strength of Steel Headed Stud Anchors in Composite Components

Where concrete breakout strength in shear is not an applicable limit state, the design shear strength, $\phi_v Q_{nv}$, and allowable shear strength, Q_{nv}/Ω_v, of one steel headed stud anchor shall be determined as:

$$Q_{nv} = F_u A_{sa} \tag{I8-3}$$

$$\phi_v = 0.65 \text{ (LRFD)} \qquad \Omega_v = 2.31 \text{ (ASD)}$$

where

A_{sa} = cross-sectional area of a steel headed stud anchor, in.^{2} (mm^2)
F_u = specified minimum tensile strength of a steel headed stud anchor, ksi (MPa)
Q_{nv} = nominal shear strength of a steel headed stud anchor, kips (N)

Where concrete breakout strength in shear is an applicable limit state, the available shear strength of one steel headed stud anchor shall be determined by one of the following:

(a) Where anchor reinforcement is developed in accordance with ACI 318 on both sides of the concrete breakout surface for the steel headed stud anchor, the minimum of the steel nominal shear strength from Equation I8-3 and the nominal strength of the anchor reinforcement shall be used for the nominal shear strength, Q_{nv}, of the steel headed stud anchor.
(b) As stipulated by the applicable building code or ACI 318 Chapter 17.

User Note: If concrete breakout strength in shear is an applicable limit state (for example, where the breakout prism is not restrained by an adjacent steel plate, flange or web), appropriate anchor reinforcement is required for the provisions of this Section to be used. Alternatively, the provisions of the applicable building code or ACI 318 Chapter 17 may be used.

3b. Tensile Strength of Steel Headed Stud Anchors in Composite Components

Where the distance from the center of an anchor to a free edge of concrete in the direction perpendicular to the height of the steel headed stud anchor is greater than or equal to 1.5 times the height of the steel headed stud anchor measured to the top of the stud head, and where the center-to-center spacing of steel headed stud anchors is greater than or equal to three times the height of the steel headed stud anchor measured to the top of the stud head, the available tensile strength of one steel headed stud anchor shall be determined as:

$$Q_{nt} = F_u A_{sa} \tag{I8-4}$$

$$\phi_t = 0.75 \text{ (LRFD)} \qquad \Omega_t = 2.00 \text{ (ASD)}$$

where

Q_{nt} = nominal tensile strength of steel headed stud anchor, kips (N)

Where the distance from the center of an anchor to a free edge of concrete in the direction perpendicular to the height of the steel headed stud anchor is less than 1.5 times the height of the steel headed stud anchor measured to the top of the stud head, or where the center-to-center spacing of steel headed stud anchors is less than three times the height of the steel headed stud anchor measured to the top of the stud head, the nominal tensile strength of one steel headed stud anchor shall be determined by one of the following:

(a) Where anchor reinforcement is developed in accordance with ACI 318 on both sides of the concrete breakout surface for the steel headed stud anchor, the minimum of the steel nominal tensile strength from Equation I8-4 and the nominal strength of the anchor reinforcement shall be used for the nominal tensile strength, Q_{nt}, of the steel headed stud anchor.
(b) As stipulated by the applicable building code or ACI 318 Chapter 17.

User Note: Supplemental confining reinforcement is recommended around the anchors for steel headed stud anchors subjected to tension or interaction of shear and tension to avoid edge effects or effects from closely spaced anchors. See the Commentary and ACI 318 for guidelines.

3c. Strength of Steel Headed Stud Anchors for Interaction of Shear and Tension in Composite Components

Where concrete breakout strength in shear is not a governing limit state, and where the distance from the center of an anchor to a free edge of concrete in the direction perpendicular to the height of the steel headed stud anchor is greater than or equal to 1.5 times the height of the steel headed stud anchor measured to the top of the stud head, and where the center-to-center spacing of steel headed stud anchors is greater than or equal to three times the height of the steel headed stud anchor measured to the top of the stud head, the nominal strength for interaction of shear and tension of one steel headed stud anchor shall be determined as:

$$\left(\frac{Q_{rt}}{Q_{ct}}\right)^{5/3}+\left(\frac{Q_{rv}}{Q_{cv}}\right)^{5/3}\leq 1.0 \qquad \text{(I8-5)}$$

where

Q_{ct} = available tensile strength, determined in accordance with Section I8.3b, kips (N)

Q_{rt} = required tensile strength, kips (N)

Q_{cv} = available shear strength, determined in accordance with Section I8.3a, kips (N)

Q_{rv} = required shear strength, kips (N)

Where concrete breakout strength in shear is a governing limit state, or where the distance from the center of an anchor to a free edge of concrete in the direction perpendicular to the height of the steel headed stud anchor is less than 1.5 times the height of the steel headed stud anchor measured to the top of the stud head, or where the center-to-center spacing of steel headed stud anchors is less than three times the height of the steel headed stud anchor measured to the top of the stud head, the nominal strength for interaction of shear and tension of one steel headed stud anchor shall be determined by one of the following:

(a) Where anchor reinforcement is developed in accordance with ACI 318 on both sides of the concrete breakout surface for the steel headed stud anchor, the minimum of the steel nominal shear strength from Equation I8-3 and the nominal strength of the anchor reinforcement shall be used for the nominal shear strength, Q_{nv}, of the steel headed stud anchor, and the minimum of the steel nominal tensile strength from Equation I8-4 and the nominal strength of the anchor reinforcement shall be used for the nominal tensile strength, Q_{nt}, of the steel headed stud anchor for use in Equation I8-5.

(b) As stipulated by the applicable building code or ACI 318 Chapter 17.

3d. Shear Strength of Steel Channel Anchors in Composite Components

The available shear strength of steel channel anchors shall be based on the provisions of Section I8.2b with the following resistance factor and safety factor:

$$\phi_v = 0.75 \text{ (LRFD)} \qquad \Omega_v = 2.00 \text{ (ASD)}$$

3e. Detailing Requirements in Composite Components

Steel anchors in composite components shall meet the following requirements:

(a) Minimum concrete cover to steel anchors shall be in accordance with ACI 318 provisions for concrete protection of headed shear stud reinforcement.

(b) Minimum center-to-center spacing of steel headed stud anchors shall be four diameters in any direction.

(c) The maximum center-to-center spacing of steel headed stud anchors shall not exceed 32 times the shank diameter.

(d) The maximum center-to-center spacing of steel channel anchors shall be 24 in. (600 mm).

User Note: Detailing requirements provided in this section are absolute limits. See Sections I8.3a, I8.3b, and I8.3c for additional limitations required to preclude edge and group effect considerations.

4. Performance Based Alternative for the Design of Shear Connection

In lieu of shear connection prescribed by, and the corresponding strength determined per, Sections I8.1 through I8.3, it shall be permitted to use an alternate form of shear connection and determine its strength through testing, provided its performance requirements are established in accordance with Sections I8.4a through I8.4d. The geometric limitations of Sections I8.1 through I8.3 do not apply to the performance evaluated by Section I8.4.

4a. Test Standard

Shear connection strength, slip capacity, and stiffness shall be established using AISI S923. An alternative test protocol may be used in the evaluation when approved by the authority having jurisdiction.

4b. Nominal and Available Strength

When determining available strength of a flexural member, the nominal tested strength of shear connection, Q_{ne}, shall be taken as 0.85 times the mean tested strength determined per Section I8.4a. When required, the design shear strength, ϕQ_{ne}, and the available shear strength, Q_{ne}/Ω, shall be determined per Section I8.3a. Alternatively, it shall be permitted to take Q_{ne} as the mean tested strength provided ϕQ_{ne} or Q_{ne}/Ω , as applicable, is determined on the basis of a reliability analysis.

User Note: An approach for establishing available strength using test data is provided in Chapter K of ANSI/AISI S100.

4c. Shear Connection Slip Capacity

The nominal shear connection slip capacity shall be taken as the average shear connection slip corresponding to each specific tested shear connection configuration. Shear connection slip capacity shall be measured at no less than 95% of the post-peak strength.

4d. Acceptance Criteria

The design using tested properties of the shear connection per Section I8.4a through I8.4c shall be limited to the geometric and material properties tested. The nominal performance characteristics are permitted to be used in design provided either the conditions (1), (2), and (3) are satisfied, or the condition (4) is, met.

(1) The maximum permitted coefficient of variation corresponding to each tested configuration of shear connection does not exceed 0.09 established over four replicate tests, or 0.15 established over nine replicate tests. It is permitted, for this purpose, to establish the number of tests using all tests of the same type of shear connection that exhibit the same failure mode.

(2) The nominal shear connection slip capacity is at least 0.25 in. (6 mm).

(3) The minimum shear elastic stiffness of the shear connection shall not be less than 2,000 kip/in. (180 N/mm).

(4) Shear connections corresponding to the values of shear connection elastic stiffness, coefficient of variation, and slip capacity other than those stipulated in conditions (1), (2), and (3) shall be deemed acceptable, provided their effect is captured in the design. In lieu of using in an analysis the shear connection elastic stiffness determined per this Section, it shall be permitted to establish the stiffness of a composite section, incorporating shear connection evaluated by this Section, directly through testing using AISI S924. When stiffness of a composite section is established using AISI S924, it shall be a mean tested value established using at least three tests.

CHAPTER J

DESIGN OF CONNECTIONS

This chapter addresses connecting elements, connectors. and the affected elements of 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 Additional Requirements for HSS and Box-Section Connections
- Appendix 3 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 and 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 B3.4b.

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

4. Compression Members with Bearing Joints

Compression members relying on bearing for load transfer shall meet the following requirements:

(a) For columns bearing on bearing plates or finished to bear at splices, there shall be sufficient connectors to hold all parts in place.

(b) For compression members other than columns finished to bear, the splice material and its connectors shall be arranged to hold all parts in line and their required strength shall be the lesser of:

(1) An axial tensile force equal to 50% of the required compressive strength of the member; or

(2) The moment and shear resulting from a transverse load equal to 2% 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 joints 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 splices in heavy sections, as defined in Sections A3.1c and A3.1d, by complete-joint-penetration (CJP) groove welds, the following provisions apply: (a) material notch-toughness requirements as given in Sections A3.1c and A3.1d; (b) weld access hole details as given in Section J1.6; (c) filler metal requirements as given in Section J2.6; and (d) thermal cut surface preparation and inspection requirements as given in Section M2.2. 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 partial-joint-penetration (PJP) groove welds on the flanges and fillet-welded web plates, or using bolts for some or all of the splice.

6. Weld Access Holes

Weld access holes shall meet the following requirements:

(a) All weld access holes required to facilitate welding operations shall be detailed to provide room for weld backing as needed.

(b) The access hole 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, nor less than 1-1/2 in. (38 mm).

(c) The access hole shall have a height not less than the thickness of the material with the access hole, nor less than 3/4 in. (19 mm), nor does it need to exceed 2 in. (50 mm).

(d) 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.

(e) In hot-rolled shapes, and built-up shapes with CJP groove welds that join the web-to-flange, weld access holes shall be free of notches and sharp reentrant corners.

(f) No arc of the weld access hole shall have a radius less than 3/8 in. (10 mm).

(g) In built-up shapes with fillet or partial-joint-penetration (PJP) groove welds that join the web-to-flange, weld access holes shall be free of notches and sharp reentrant corners.

(h) The access hole is 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.

(i) For heavy shapes, as defined in Sections A3.1c and A3.1d, the thermally cut surfaces of weld access holes shall be ground to bright metal.

(j) If the curved transition portion of weld access holes is formed by predrilled or sawed holes, that portion of the access hole need not be ground.

7. Placement of Welds and Bolts

Groups of welds or bolts at the ends of any member that 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 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 in the design of shear connections on a common faying surface where strain compatibility between the bolts and welds is considered.

It is permitted to determine the available strength, ϕR_n and R_n/Ω, as applicable, of a joint combining the strengths of high-strength bolts and longitudinal fillet welds as the sum of (1) the nominal slip resistance, R_n, for bolts as defined in Equation J3-4 according to the requirements of a slip-critical connection and (2) the nominal weld strength, R_n, as defined in Section J2.4, when the following apply:

(a) $\phi = 0.75$ (LRFD); $\Omega = 2.00$ (ASD) for the combined joint.

(b) When the high-strength bolts are pretensioned according to the requirements of Table J3.1 or Table J3.1M, using the turn-of-nut or combined method, the longitudinal fillet welds shall have an available strength of not less than 50% of the required strength of the connection.

(c) When the high-strength bolts are pretensioned according to the requirements of Table J3.1 or Table J3.1M, using any method other than the turn-of-nut method, the longitudinal fillet welds shall have an available strength of not less than 70% of the required strength of the connection.

(d) The high-strength bolts shall have an available strength of not less than 33% of the required strength of the connection.

In joints with combined bolts and longitudinal welds, the strength of the connection need not be taken as less than either the strength of the bolts alone or the strength of the welds alone.

9. Welded Alterations to Structures with Existing Rivets or Bolts

In making welded alterations to structures, existing rivets and high-strength bolts in standard or short-slotted holes transverse to the direction of load, and tightened to the requirements of slip-critical connections are permitted to be utilized for resisting loads present at the time of alteration, and the welding need only provide the additional required strength. The weld available strength shall provide the additional required strength, but not less than 25% of the required strength of the connection.

User Note: The provisions of this section are generally recommended for alteration in building designs or for field corrections. Use of the combined strength of bolts and welds on a common faying surface is not recommended for new design.

10. High-Strength Bolts in Combination with Rivets

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.

J2. WELDS AND WELDED JOINTS

All provisions of the *Structural Welding Code—Steel* (AWS D1.1/D1.1M), hereafter referred to as AWS D1.1/D1.1M, apply under this Specification, with the exception that the provisions of the listed Specification sections apply under this Specification in lieu of the cited AWS provisions as follows:

(a) Section J1.6 in lieu of AWS D1.1/D1.1M clause 5.16
(b) Section J2.2a in lieu of AWS D1.1/D1.1M clauses 2.4.2.10 and 2.4.4.4
(c) Table J2.2 in lieu of AWS D1.1/D1.1M Table 2.1
(d) Table J2.5 in lieu of AWS D1.1/D1.1M Table 2.3
(e) Appendix 3, Table A-3.1 in lieu of AWS D1.1/D1.1M Table 2.5
(f) Section B3.11 and Appendix 3 in lieu of AWS D1.1/D1.1M clause 2, Part C
(g) Section M2.2 in lieu of AWS D1.1/D1.1M clauses 5.14 and 5.15

1. Groove Welds

1a. Effective Area

The effective area of groove welds shall be taken as the length of the weld times the effective throat.

The effective throat of a CJP groove weld shall be the thickness of the thinner part joined.

When filled flush to the surface, the effective weld throat for a PJP groove weld shall be as given in Table J2.1 and the effective weld throat for a flare groove weld shall be as given in Table J2.2. The effective throat of a PJP groove weld or flare groove weld filled less than flush shall be as shown in Table J2.1 or Table J2.2, less the greatest perpendicular dimension measured from a line flush to the base metal surface to the weld surface.

User Note: The effective throat of a PJP groove weld is dependent on the process used and the weld position. The design 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.

For PJP groove welds effective throats larger than those for prequalified PJP groove welds in AWS D1.1/D1.1M Figure 3.2 and flare groove welds in Table J2.2 are permitted for a given welding procedure specification (WPS), provided the fabricator establishes by testing the consistent production of such larger effective throat. Testing shall consist of sectioning the weld normal to its axis, at mid-length, and at 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. During production of welds with increased effective throats, single pass and the root pass of multi-pass welds shall be made using a mechanized, automatic, or robotic process, with no decrease in current or increase in travel speed from that used for testing.

1b. Limitations

The minimum effective throat 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.

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

Welding Process	**Welding Position** F (flat), H (horizontal), V (vertical), OH (overhead)	**Groove Type** (AWS D1.1, Figure 3.3)	**Effective Throat**
Shielded metal arc (SMAW)	All	J or U groove	depth of groove
Gas metal arc (GMAW) Flux cored arc (FCAW)		60° V	
Submerged arc (SAW)	F	J or U groove 60° bevel or V	
Gas metal arc (GMAW)	F, H	45° bevel	depth of

Flux cored arc (FCAW)			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		

TABLE J2.2
Effective Throat of
Flare Groove Welds

Welding Process	Flare Bevel Groove[a]	Flare V-Groove
GMAW and FCAW-G	5/8R	3/4R
SMAW and FCAW-S	5/16R	5/8R
SAW	5/16R	1/2R

[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 (is permitted to be assumed equal to 2t for HSS)

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

Material Thickness of Thinner Part Joined, in. (mm)	Minimum Effective Throat,[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.

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 a given welding procedure specification (WPS), provided the fabricator establishes by testing the consistent production of such larger effective throat. Testing shall consist of sectioning the weld normal to its axis, at mid-length, and terminal ends. During production, single pass welds and the root pass of multi-pass welds shall be made using a mechanized, automatic or robotic process, with no decrease in current or increase in travel speed from that used for testing.

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

Fillet welds shall meet the following limitations:

(a) 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 limitations do not apply to fillet weld reinforcements of 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. When non-low hydrogen electrodes are used single pass welds must be used.
Note: See Section J2.2b for maximum size of fillet welds.

(b) The maximum size of fillet welds of connected parts shall be:

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

(2) 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 design and fabrication documents 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.

(c) The minimum length of fillet welds designed on the basis of strength shall be not less than four times the nominal weld size, or else the effective size of the weld shall not be taken to exceed one-quarter of its length.

(d) The effective length of fillet welds shall be determined as follows:

(1) For end-loaded fillet welds with a length up to 100 times the weld size, it is permitted to take the effective length equal to the actual length.

(2) 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, determined as:

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

where
l = actual length of end-loaded weld, in. (mm)

w = size of weld leg, in. (mm)

(3) When the length of the weld exceeds 300 times the leg size, w, the effective length shall be taken as $180w$.

User Note: For the effect of longitudinal fillet weld length in end connections upon the effective area of the connected member see Section D3.

(e) Intermittent fillet welds are permitted to be used to transfer calculated stress across a joint or faying surfaces and to join components of built-up members. The 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).

(f) 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.

(g) Fillet weld terminations shall be detailed in a manner that does not result in a notch in the base metal subject to applied tension loads. Components shall not be connected by welds where the weld would prevent the deformation required to provide assumed design conditions.

User Note: Fillet weld terminations should be detailed in a manner that does not result in a notch in the base metal transverse to applied tension loads that can occur as a result of normal fabrication. An accepted practice to avoid notches in base metal is to stop fillet welds short of the edge of the base metal by a length approximately equal to the size of the weld. In most welds, the effect of stopping short can be neglected in strength calculations.

There are two common details where welds are terminated short of the end of the joint to permit relative deformation between the connected parts:

- Welds on the outstanding legs of beam clip-angle connections are returned on the top of the outstanding leg and stopped no more than 4 times the weld size and not greater than half the leg width from the outer toe of the angle.
- Fillet welds connecting transverse stiffeners to webs of girders that are ¾ in. thick or less are stopped 4 to 6 times the web thickness from the web toe of the flange-to web fillet weld, except where the end of the stiffener is welded to the flange.

Details of fillet weld terminations may be shown on shop standard details.

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

(i) For fillet welds in slots, 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.

3. Plug and Slot Welds

3a. Effective Area

The effective shear area of plug and slot welds shall be taken as the nominal 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 or separation of lapped parts, and to join component parts of built-up members, subject to the following limitations:

(a)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.

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

(c) The length of slot for a slot weld shall not exceed 10 times the thickness of the weld.

(d) 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.

(e) 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.

(f) The minimum spacing of lines of slot welds in a direction transverse to their length shall be four times the width of the slot.

(g) The minimum center-to-center spacing in a longitudinal direction on any line shall be two times the length of the slot.

(h) 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

(a) The design strength, ϕR_n and the allowable strength, R_n/Ω, of welded joints shall be the lower value of the base material strength determined according to the limit states of tensile rupture and shear rupture and the weld metal strength determined according to the limit state of rupture as follows:

For the base metal

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

For the weld metal

$$R_n = F_{nw} A_{we} \tag{J2-3}$$

where

A_{BM} = area of the base metal, in.^{2} (mm^2)
A_{we} = effective area of the weld, in.^{2} (mm^2)
F_{nBM} = nominal stress of the base metal, ksi (MPa)
F_{nw} = nominal stress of the weld metal, ksi (MPa)

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

TABLE J2.5
Available Strength of Welded Joints, ksi (MPa)

Load Type and Direction Relative to Weld Axis	Pertinent Metal	ϕ and Ω	Nominal Stress (F_{nBM} or F_{nw}), ksi (MPa)	Effective Area (A_{BM} or A_{we}), 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 is permitted to be neglected 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 V-GROOVE AND FLARE BEVEL GROOVE WELDS					
Tension– Normal to weld axis	Base	$\phi = 0.75$ $\Omega = 2.00$	F_u	See J4	
	Weld	$\phi = 0.80$ $\Omega = 1.88$	$0.60 F_{EXX}$	See J2.1a	

Compression– Connections of Members designed to bear as described in Section J1.4(b)	Compressive stress is permitted to be neglected in design of welds joining the parts.				
Compression– Connections not designed to bear	Base	ϕ = 0.90 Ω = 1.67	F_y	See J4	
	Weld	ϕ = 0.80 Ω = 1.88	0.90 F_{EXX}	See J2.1a	
Tension or compression– Parallel to weld axis	Tension or compression in parts joined parallel to a weld is permitted to be neglected in design of welds joining the parts.				Filler metal with a strength level equal to or less than matching filler metal is permitted.
Shear	Base	Governed by J4			
	Weld	ϕ = 0.75 Ω = 2.00	0.60 F_{EXX}	See J2.1a	
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	ϕ = 0.75 Ω = 2.00	0.60 F_{EXX} [d]	See J2.2a	
Tension or compression– Parallel to weld axis	Tension or compression in parts joined parallel to a weld is permitted to be neglected 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	ϕ = 0.75 Ω = 2.00	$0.60F_{EXX}$	J2.3a	

[a] For matching weld metal, see AWS D1.1/D1.1M clause 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 are permitted to be used for CJP 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, where ϕ = 0.80, Ω = 1.88 and 0.60 F_{EXX} is the nominal strength.
[d] The provisions of Section J2.4(b) are also applicable.

(b) The design strength, ϕR_n and the allowable strength, R_n/Ω, of weld groups shall be determined as follows:

(1) For a linear weld group with a uniform leg size, loaded through the center of gravity, the available strength is permitted to be determined accounting for a directional strength increase if strain compatibility of the various weld elements is considered, using

$$F_{nw} = 0.60F_{EXX}\left(1.0+0.50\sin^{1.5}\theta\right) \quad \text{(J2-4)}$$

where

F_{EXX} = filler metal classification strength, ksi (MPa)

ϕ = 0.75 (LRFD); Ω = 2.00 (ASD)

θ = angle between the line of action of the required force and the weld longitudinal axis, degrees

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

(2) For fillet welds to the ends of rectangular HSS where the weld ia loaded in tension, the directional strength increase is not applicable. Hence,

$$F_{nw} = 0.60F_{EXX} \quad \text{(J2-5)}$$

(3) For fillet weld groups concentrically loaded and consisting of elements with a uniform leg size 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 the following:

(i) $R_n = R_{nwl} + R_{nwt}$ (J2-6a)

or

(ii) $R_n = 0.85\,R_{nwl} + 1.5R_{nwt}$ (J2-6b)

where

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

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

User Note: The instantaneous center method is a valid way to calculate the strength of weld groups consisting of weld elements in various directions based on strain compatibility.

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 filler metal for use with CJP groove welds subject to tension normal to the effective area shall comply with the requirements for matching filler metals given in AWS D1.1/D1.1M.

User Note: The following User Note Table summarizes the AWS D1.1/D1.1M provisions for matching filler metals. Other restrictions exist. For a complete list of base metals and prequalified matching filler metals, see AWS D1.1/D1.1M Table 3.1 and Table 3.2.

Base Metal (ASTM)	Matching Filler Metal
A36 ≤ 3/4 in. thick	60- and 70-ksi filler metal
A36 > 3/4 in. thick, A588[a], A1011, A572 Gr. 50 and 55, A913 Gr. 50, A992, A1018	SMAW: E7015, E7016, E7018, E7028 Other processes: 70-ksi filler metal
A913 Gr. 60 and 65	80-ksi filler metal
A913 Gr. 70	90-ksi filler metal

[a] For corrosion resistance and color similar to the base metal, see AWS D1.1/D1.1M clause 3.7.3.

Notes:
In joints with base metals of different strengths, 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 may be used when matching strength is required.

Filler metal with a specified minimum Charpy V-notch toughness of 20 ft-lb (27 J) at 40°F (4°C) or lower shall be used in the following joints:

(a) CJP 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 groove weld
(b) CJP groove welded splices subject to tension normal to the effective area in heavy sections, as defined in Sections 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, THREADED PARTS and BOLTED CONNECTIONS

1. Common Bolts

ASTM A307 bolts are permitted except where pretensioning is specified.

2. High-Strength Bolts

Use of high-strength bolts and bolting components shall conform to the provisions of the *Specification for Structural Joints Using High-Strength Bolts*, hereafter referred to as the RCSC *Specification*, except as modified by this Specification. Major modifications include the following:

(a) This Specification allows bolt grades of ASTM F3125 Grades A325, A325M, A490, A490M, F1852 F2280 ASTM F3043, F3111, F3148, A354 Grade BC, A354 Grade BD, and A449 bolts.
(b) The RCSC *Specification* Section 5.2 is replaced with Section J3.7 of this *Specifiction*
(c) Exceptions to the RCSC *Specification* associated with cyclically loaded connections are contained in Appendix 3 of this Specification.
(d) Water-jet cutting of holes is permitted by Section M2.5 of this Specification.

High-strength bolts in this Specification are grouped according to material strength as follows:

Group 120—ASTM F3125/F3125M Grades A325, A325M, F1852 and ASTM A354 Grade BC
Group 144 ASTM F3148 Grade 144
Group 150—ASTM F3125/F3125M Grades A490, A490M, F2280 and ASTM A354 Grade BD
Group 200 ASTM F3043 and F3111

Use of Group 144 bolting assemblies shall conform to the provisions of ASTM F3148. Assemblies may be used in snug-tight, pretensioned or slip-critical connections, using the installation procedures provided in ASTM F3148 and RCSC Section 8.2.5.

Use of Group 200 high-strength bolting assemblies shall conform to the applicable provisions of their ASTM standard. ASTM F3043 and F3111 Grade 1 assemblies may be installed only to the snug-tight condition. ASTM F3043 and F3111 Grade 2 assemblies may be used in snug-tight, pretensioned and slip-critical connections, using procedures provided in the applicable ASTM standard.

User Note: The use of Group 200 bolting assemblies is limited to specific building locations and noncorrosive environmental conditions by the applicable ASTM standard.

When assembled, all joint surfaces, including those adjacent to the washers, shall be free of scale, except tight mill scale.

(a) Bolting assemblies are permitted to be installed to the snug-tight condition when used in:

  (1) Bearing-type connections, except as stipulated in Section E6
  (2) Tension or combined shear and tension applications, for Group 120 bolts only, where loosening or fatigue due to vibration or load fluctuations are not design considerations

(b) Bolts in the following connections shall be pretensioned:

(1) As required by the RCSC *Specification*
(2) Connections subjected to vibratory loads where bolt loosening is a consideration
(3) End connections of built-up members composed of two shapes either interconnected by bolts, or with at least one open side interconnected by perforated cover plates or lacing with tie plates, as required in Section E6.1

(c) The following connections shall be designed as slip critical:

(1) As required by the RCSC *Specification*
(2) The extended portion of bolted, partial-length cover plates, as required in Section F13.3

The snug-tight condition is defined in the RCSC *Specification*. Bolts to be tightened to a condition other than snug tight shall be clearly identified on the design documents. (See Table J3.1 or J3.1M for minimum bolt pretension for connections designated as pretensioned or slip critical.)

User Note: There are no specific minimum or maximum tension requirements for snug-tight bolts. Bolts that have been pretensioned are permitted in snug-tight connections unless specifically prohibited on design documents.

When bolt requirements cannot be provided within the RCSC *Specification* limitations because of requirements for lengths exceeding 12 diameters or diameters exceeding 1-1/2 in. (38 mm), bolts or threaded rods conforming to Group 120 or Group 150 materials are permitted to be used in accordance with the provisions for threaded parts in Table J3.2.

When ASTM A354 Grade BC, A354 Grade BD, or A449 bolts and threaded rods are used in pretensioned connections, the bolt geometry, including the thread pitch, thread length, head and nut(s), shall be equal to or (if larger in diameter) proportional to that required by the RCSC *Specification*. 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 following requirements apply for bolted connections:

(a) The nominal dimensions of standard, oversized, short-slotted and long-slotted holes for bolts are given in Table J3.3 or Table J3.3M.

User Note: Bolt holes with a smaller nominal diameter are permitted. See RCSC Table 3.1 for bolt hole fabrication tolerances. See Section J9 for diameters of holes in base plates for anchor rods providing anchorage to concrete.

(b) 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.

(c) 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.

TABLE J3.1
Minimum Bolt Pretension, kips

Bolt Size, in.	Group 120[a]	Group 144[a] [b] And Group 150[b]	Group 200, Grade 2[c]
1/2	12	15	–
5/8	19	24	–
3/4	28	35	–
7/8	39	49	–
1	51	64	90
1 1/8	64	80	113
1-1/4	81	102	143
1 3/8	97	121	–
1-1/2	118	148	–

[a] Equal to 0.70 times the minimum tensile strength of bolts as specified in ASTM F3125/F3125M for Grade A325 rounded off to nearest kip.
[b] Equal to 0.70 times the minimum tensile strength of bolts as specified in ASTM F3125/F3125M for Grade A490 rounded off to nearest kip. Group 144 (F3148) assemblies have the same specified minimum pretension as Group 150.
[c] Equal to 0.70 times the minimum tensile strength of bolts as specified in ASTM F3043 and F3111 for Grade 2, rounded off to nearest kip,.

TABLE J3.1M
Minimum Bolt Pretension,(metric) kN

Bolt Size, mm	Group 120 [a]	Group 150[b]
M16	91	114
M20	142	179
M22	176	221
M24	205	257
M27	267	334
M30	326	408
M36	475	595

[a]Equal to 0.70 times the minimum tensile strength of bolts as specified in ASTM F3125/F3125M for Grade A325M, rounded off to nearest kN.
[a]Equal to 0.70 times the minimum tensile strength of bolts as specified in ASTM F3125/F3125M for Grade A490M bolts rounded off to nearest kN.

User Note: Metric grades manufactured to F3125 Grade A325M and A490M are similar to Group 120 (830MPa) and Group 150 (1030MPa), respectively.

(d) Oversized holes are permitted in any or all plies of slip-critical connections, but they shall not be used in bearing-type connections.

(e) 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 loading in bearing-type connections.

(f) 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 loading in bearing-type connections.

(g) Washers shall be provided in accordance with the RCSC *Specification* Section 6, except for Group 200 bolting assemblies, washers shall be provided in accordance with the applicable ASTM standard.

User Note: When Group 200 heavy-hex bolting assemblies are used, a single washer is used under the bolt head and a single washer is used under the nut. When Group 200 twist-off bolting assemblies are used, a single washer is used under the nut. Washers are of the type specified in the ASTM standard for the bolting assembly.

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. However, the clear distance between bolt holes or slots shall not be less than d.

User Note: A distance between centers of standard, oversize or slotted holes of $3d$ is preferred.

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

Description of Fasteners	Nominal Tensile Stress, F_{nt}, ksi (MPa)[a][b]	Nominal Shear Stress in Bearing-Type Connections, F_{nv}, ksi (MPa) [c]	
		Threads Not Excluded from Shear Planes – (N)	Threads Excluded from Shear Planes – (X)
A307 bolts	45 (310)[c]	27 (186)[c] [d]	27 (186)[c][d]
Group 120 (e.g., A325)	90 (620)	54 (372)	68 (469)
Group 144 (e.g., F3148)	108 (745)	65 (448)	81 (565)
Group 150 (e.g., A490)	113 (780)	68 (469)	84 (579)
Group 200 (e.g., F3043)	150 (1040)	90 (620)[e]	113 (779)[e]
Threaded parts meeting the requirements of Section A3.4,	0.75 F_u	0.450 F_u	0.563 F_u

[a] For high-strength bolts subject to tensile fatigue loading, see Appendix 3.
[b] For nominal tensile strength it is permitted to use the tensile stress areaof the threaded rod or bolt multiplied by the minimum specified tensile stress of the rod or bolt material, in lieu of the tabulated values based on a nomnal tensile stress ares of 0.75 times the gross area
[c] For end loaded connections with a fastener pattern length greater than 38 in. (950 mm), F_{nv} shall be reduced to 83.3% of the tabulated values. Fastener pattern length is the maximum distance parallel to the line of force between the centerline of the bolts connecting two parts with one faying surface.
d] For A307 bolts, the tabulated values shall be reduced by 1% for each 1/16 in. (2 mm) over five diameters of length in the grip.
[e] Threads assumed and permitted in shear planes in all cases.
[e] The transition area of Group 200 bolts is considered part of the threaded section.

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 Table 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 Table 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 hole to the nearest edge of elements in contact shall be 12 times the thickness of the connected element under consideration, but shall not exceed 6 in. (150 mm). The longitudinal spacing of bolt holes between elements consisting of a plate and a shape, or two plates, in continuous contact shall be as follows:

(a) For painted members or unpainted members not subject to corrosion, the spacing shall not exceed 24 times the thickness of the thinner part or 12 in. (300 mm).

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

User Note: The dimensions in (a) and (b) do not apply to elements consisting of two shapes in continuous contact.

TABLE J3.3
Nominal Hole Dimensions, in.

Bolt Diameter	Hole Dimensions			
	Standard (Dia.)	Oversize (Dia.)	Short-Slot (Width x Length)	Long-Slot (Width x Length)
1/2	9/16	5/8	9/16 x 11/16	9/16 x 1-1/4
5/8	11/16	13/16	11/16 x 7/8	11/16 x 1-9/16
3/4	13/16	15/16	13/16 x 1	13/16 x 1-7/8
7/8	15/16	1-1/16	15/16 x 1-1/8	15/16 x 2-3/16
1	1-1/8	1-1/4	1-1/8 x 1-5/16	1-1/8 x 2-1/2
≥1-1/8	*d* + 1/8	*d* + 5/16	(*d* + 1/8) x (*d* + 3/8)	(*d* + 1/8) x 2.5*d*

TABLE J3.3M
Nominal Hole Dimensions, mm

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

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

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

Bolt Diameter	Minimum Edge Distance
1/2	3/4
5/8	7/8
3/4	1
7/8	1-1/8
1	1-1/4
1-1/8	1-1/2
1-1/4	1-5/8
Over 1-1/4	1-1/4d

[a] If necessary, lesser edge distances are permitted provided the applicable provisions from Sections J3.10 and J4 are satisfied, but edge distances less than one bolt diameter are not permitted without approval from the engineer of record.
[b] For oversized or slotted holes, see Table J3.5.

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

Bolt Diameter	Minimum Edge Distance
16	22
20	26
22	28
24	30
27	34
30	38
36	46
Over 36	1.25d

[a] If necessary, lesser edge distances are permitted provided the applicable provisions from Sections J3.10 and J4 are satisfied, but edge distances less than one bolt diameter are not permitted without approval from the engineer of record.
[b] For oversized or slotted holes, see Table J3.5M.

6. Tensile and Shear Strength of Bolts and Threaded Parts

The design tensile or shear strength, ϕR_n, and the allowable tensile 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 tension rupture and shear rupture as:

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

$\phi = 0.75$ (LRFD) $\quad\quad$ $\Omega = 2.00$ (ASD)

where

A_b = nominal unthreaded body area of bolt or threaded part, in.^{2} (mm^2)

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

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

User Note: The available strength of a bolt in shear depends on whether the bolt is sheared through its shank or through the threads / thread runout. Bolts that are relatively short may be produced as fully threaded, without a shank, and thus may not be able to be installed in the “threads excluded” condition.

User Note: The force that can be resisted by a snug-tightened or pretensioned high-strength bolt or threaded part may be limited by the bearing or tearout strength at the bolt hole per Section J3.10. The effective strength of an individual fastener may be taken as the lesser of the fastener shear strength per Section J3.6 or the bearing or tearout strength at the bolt hole per Section J3.10. The strength of the bolt group is taken as the sum of the effective strengths of the individual fasteners.

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:

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

$\phi = 0.75$ (LRFD) $\quad\quad$ $\Omega = 2.00$ (ASD)

where

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

$$1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_{rv} \le F_{nt} \quad \text{(LRFD)} \tag{J3-3a}$$

$$1.3F_{nt} - \frac{\Omega F_{nt}}{F_{nv}} f_{rv} \le F_{nt} \quad \text{(ASD)} \tag{J3-3b}$$

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

F_{nv} = nominal shear stress from Table J3.2, ksi (MPa)
f_{rv} = required shear stress using LRFD or ASD load combinations, ksi (MPa)

The available shear stress of the fastener shall equal or exceed the required shear stress, f_{rv}.

User Note: Note that when the required stress, f, in either shear or tension, is less than or equal to 30% 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.

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

Nominal Diameter of Fastener	Oversized Holes	Slotted Holes		
		Long Axis Perpendicular to Edge		Long Axis Parallel to Edge
		Short Slots	Long Slots[a]	
≤ 7/8	1/16	1/8	3/4d	0
1	1/8	1/8		
≥1 1/8	1/8	3/16		

[a] When the length of the slot is less than the 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	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 the length of the slot is less than the 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.

8. High-Strength Bolts in Slip-Critical Connections

Slip-critical connections shall be designed to prevent slip and for the limit states of bearing-type connections. When slip-critical bolts pass through fillers, all surfaces subject to slip shall be prepared to achieve design slip resistance.

The single bolt available slip resistance for the limit state of slip shall be determined as follows:

$$R_n = \mu D_u h_f T_b n_s \qquad \text{(J3-4)}$$

(a) For standard size and short-slotted holes perpendicular to the direction of the load

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

(b) For oversized and short-slotted holes parallel to the direction of the load

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

(c) For long-slotted holes

$$\phi = 0.70 \text{ (LRFD)} \qquad \Omega = 2.14 \text{ (ASD)}$$

where

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 are permitted if approved by the engineer of record.

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

h_f = factor for fillers, determined as follows:

(1) For one filler between connected parts

$$h_f = 1.0$$

(2) For two or more fillers between connected parts

$$h_f = 0.85$$

n_s = number of slip planes required to permit the connection to slip

μ = mean slip coefficient for Class A or B surfaces, as applicable, and determined as follows, or as established by tests:

(1) For Class A surfaces (unpainted clean mill scale steel surfaces or surfaces with Class A coatings on blast-cleaned steel or hot-dipped galvanized and roughened surfaces)

$$\mu = 0.30$$

(2) For Class B surfaces (unpainted blast-cleaned steel surfaces or surfaces with Class B coatings on blast-cleaned steel)

$$\mu = 0.50$$

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_{sc}, determined as follows:

$$k_{sc} = 1 - \frac{T_u}{D_u T_b n_b} \geq 0 \quad \text{(LRFD)} \qquad \text{(J3-5a)}$$

$$k_{sc} = 1 - \frac{1.5 T_a}{D_u T_b n_b} \geq 0 \quad \text{(ASD)} \qquad \text{(J3-5b)}$$

where

T_a = required tension force using ASD load combinations, kips (kN)
T_u = required tension force using LRFD load combinations, kips (kN)
n_b = number of bolts carrying the applied tension

10. Bearing and Tearout Strength at Bolt Holes

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

$\phi = 0.75$ (LRFD) $\Omega = 2.00$ (ASD)

The nominal strength of the connected material, R_n, is determined as follows:

(a) For connections with plies in contact:

1. The strength of a connected element at 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 shall be the lesser of:

(a) Bearing

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

$$R_n = 2.4dtF_u \tag{J3-6a}$$

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

$$R_n = 3.0dtF_u \tag{J3-6b}$$

(b) Tearout

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

$$R_n = 1.2l_ctF_u \tag{J3-6c}$$

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

$$R_n = 1.5l_ctF_u \tag{J3-6d}$$

2. The strength of a connected element at a bolt in a connection with long-slotted holes with the slot perpendicular to the direction of force is the lesser of:

(a) Bearing

$$R_n = 2.0dtF_u \tag{J3-6e}$$

(b) Tearout

$$R_n = 1.0 l_c t F_u \quad \text{(J3-6f)}$$

(b) For connections made using bolts or rods that pass completely through an unstiffened box member or HSS

(1) Bearing shall satisfy Section J7 and Equation J7-1

(2) Tearout

(i) For a bolt in a connection with a standard hole or a short-slotted hole with the slot perpendicular to the direction of force:

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

$$R_n = 1.2 l_c t F_u \quad \text{(J3-6g)}$$

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

$$R_n = 1.5 l_c t F_u \quad \text{(J3-6h)}$$

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

$$R_n = 1.0 l_c t F_u \quad \text{(J3.6i)}$$

where

F_u = specified minimum tensile strength of the connected material, ksi (MPa)

d = nominal fastener diameter, in. (mm)

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)

Bearing strength and tearout 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. Wall Strength at 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

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

$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)

(b) For tensile rupture of connecting elements

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

$\phi = 0.75$ (LRFD) $\Omega = 2.00$ (ASD)

where

A_e = effective net area as defined in Section D3, in.^{2} (mm^2)

User Note: The effects of shear lag or concentrated loads dispersed within the element may cause only a portion of the area to be effective in resisting the load. For shear lag see Chapter D

2. Strength of Elements in Shear

The available shear 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

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

$\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)

where

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

(b) For shear rupture of the element

$$R_n = 0.60 F_u A_{nv} \tag{J4-4}$$

$\phi = 0.75$ (LRFD) $\Omega = 2.00$ (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 paths and a perpendicular tension failure path shall be determined as follows:

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \leq 0.60F_yA_{gv} + U_{bs}F_uA_{nt} \tag{J4-5}$$

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

where

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

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

User Note: Typical cases where U_{bs} should be taken equal to 0.5 are illustrated in the Commentary

For connections with round holes, A_{ev} is calculated with a shear length reduction for each hole in the shear plane, l_{vh}, according to Equation J4-6. For connections with slotted holes, l_{vh} is the nominal slot dimension parallel to the shear plane plus 1/16 in.

$$l_{vh} = \sqrt{d_h^2 - d^2} \tag{J4-6}$$

where

A_{ev} = effective area subjected to shear, in.^{2} (mm^2)
A_{nt} = net area subjected to tension, in.^{2} (mm^2)
d = bolt diameter, in.
d_h = nominal hole diameter plus 1/16 in. (2 mm), in.

For concentrically loaded connections that are symmetrical about the loading axis and beam webs at welded clip angle connections:

$U_{bs} = 1.0$

For end connections at axially loaded angles and axially loaded tee-shape members bolted through the web, and for outstanding legs of clip angles in single-angle connections:

$U_{bs} = 0.70$ for connections with one bolt row parallel to the load
$U_{bs} = 0.50$ for connections with two bolt rows parallel to the load

For beam webs at bolted end connections subjected to vertical shear and for connecting elements at these interfaces:

$U_{bs} = 0.85$ for connections with one vertical bolt row
$U_{bs} = 0.40$ for connections with two vertical bolt rows
$U_{bs} = 0.50$ for connections with two bolt rows parallel to the load

For beam webs at bolted end connections subjected to vertical shear and for connecting elements at these interfaces:

U_{bs} = 0.85 for connections with one vertical bolt row
U_{bs} = 0.40 for connections with two vertical bolt rows

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:

(a) When $L_c/r \le 25$

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

$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)

(b) When $L_c/r > 25$, the provisions of Chapter E apply;

where
L_c = KL = effective length, in. (mm)
K = effective length factor
L = laterally unbraced length of the member, in. (mm)

User Note: The effective length factors used in computing compressive strengths of connecting elements are specific to the end restraint provided and may not necessarily be taken as unity when the direct analysis method is employed.

5. Strength of Elements in Flexure

The available flexural strength of affected elements shall be the lower value obtained according to the limit states of flexural yielding, local buckling, flexural lateral-torsional buckling, and flexural rupture.

J5. FILLERS

1. Fillers in Welded Connections

Whenever it is necessary to use fillers in joints required to transfer applied force, the fillers and the connecting welds shall conform to the requirements of Section J5.1a or Section J5.1b, as applicable.

1a. Thin Fillers

Fillers less than 1/4 in. (6 mm) thick shall not be used to transfer stress. When the thickness of the fillers is less than 1/4 in. (6 mm), or when the thickness of the filler is 1/4 in. (6 mm) or greater but not sufficient to transfer the applied force between the connected parts, the filler shall be kept flush with the edge of the outside connected part, and the size of the weld shall be increased over the required size by an amount equal to the thickness of the filler.

1b. Thick Fillers

When the thickness of the fillers is sufficient to transfer the applied force between the connected parts, the filler shall extend beyond the edges of the outside connected base metal. The welds joining the outside connected base metal to the filler shall be sufficient to transmit the force to the filler and the region subjected to the applied force in the filler shall be sufficient to prevent overstressing the filler. The welds joining the filler to the inside connected base metal shall be sufficient to transmit the applied force.

2. Fillers in Bolted Bearing-Type Connections

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:

(a) The shear strength of the bolts shall be multiplied by the factor

$$1 - 0.4(t - 0.25)$$

$$1 - 0.0154(t - 6) \qquad \text{(S.I.)}$$

but not less than 0.85, where t is the total thickness of the fillers.

(b) The fillers shall be welded or extended beyond the joint and bolted to uniformly distribute the total force in the connected element over the combined cross section of the connected element and the fillers.

(c) The size of the joint shall be increased to accommodate a number of bolts that is equivalent to the total number required in (b).

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, shall be determined as follows:

(a) For finished surfaces, pins in reamed, drilled, or bored holes, bolts or rods that pass completely through an unstiffened box or HSS member, and ends of fitted bearing stiffeners

$$R_n = 1.8F_y A_{pb} \qquad \text{(J7-1)}$$

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

(b) For expansion rollers and rockers

(1) When $d \le 25$ in. (630 mm)

$$R_n = \frac{1.2\left(F_y - 13\right) l_b d}{20} \quad \text{(J7-2)}$$

$$R_n = \frac{1.2\left(F_y - 90\right) l_b d}{20} \quad \text{(J7-2M)}$$

(2) When $d > 25$ in. (630 mm)

$$R_n = \frac{6.0\left(F_y - 13\right) l_b \sqrt{d}}{20} \quad \text{(J7-3)}$$

$$R_n = \frac{30.2\left(F_y - 90\right) l_b \sqrt{d}}{20} \quad \text{(J7-3M)}$$

where

d = diameter, in. (mm)

l_b = length of bearing, in. (mm)

J8. COLUMN BASES AND BEARING ON CONCRETE

Provisions 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:

$\phi_c = 0.65$ (LRFD) $\quad\quad \Omega_c = 2.31$ (ASD)

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

(a) On the full area of a concrete support

$$P_p = 0.85 f_c' A_1 \quad \text{(J8-1)}$$

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

$$P_p = 0.85 f_c' A_1 \sqrt{A_2 / A_1} \le 1.7 f_c' A_1 \quad \text{(J8-2)}$$

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)

f_c' = specified compressive strength of concrete, ksi (MPa)

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 resulting 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.

Design of anchor rods for the transfer of forces to the concrete foundation shall satisfy the requirements of ACI 318 (ACI 318M) or ACI 349 (ACI 349M).

User Note: Column bases should be designed considering bearing against concrete elements, including when columns are required to resist a horizontal force at the base plate. See AISC Design Guide 1, *Base Plate and Anchor Rod Design*, Second Edition, for column base design information.

When anchor rods are used to resist horizontal forces, hole size, anchor rod setting tolerance, and the horizontal movement of the column shall be considered in the design.

Larger oversized holes and slotted holes are permitted in base plates when adequate bearing is provided for the nut by using ASTM F844 washers or plate washers to bridge the hole.

User Note: The recommended hole sizes, corresponding washer dimensions and nuts are given in the AISC *Steel Construction Manual* and ASTM F1554. ASTM F1554 anchor rods may be furnished in accordance with product specifications with a body diameter less than the nominal diameter. Load effects such as bending and elongation should be calculated based on minimum diameters permitted by the product specification. See ASTM F1554 and the table, "Applicable ASTM Specifications for Various Types of Structural Fasteners," in Part 2 of the AISC *Steel Construction Manual*.

User Note: See ACI 318 (ACI 318M) 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 is 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, Section 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.

> **User Note:** Design guidance for members other than wide-flange sections and similar built-up shapes, including HSS members can be found in the Commentary.

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:

$$R_n = 6.25 F_{yf} t_f^2 \quad \text{(J10-1)}$$

$\phi = 0.90$ (LRFD) $\quad\quad$ $\Omega = 1.67$ (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%.

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:

$\phi = 1.00$ (LRFD) $\quad\quad$ $\Omega = 1.50$ (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 full nominal depth of the member, d,

$$R_n = F_{yw} t_w \left(5k + l_b\right) \quad \text{(J10-2)}$$

(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 full nominal depth of the member, d,

$$R_n = F_{yw} t_w \left(2.5k + l_b\right) \quad \text{(J10-3)}$$

where

F_{yw} = specified minimum yield stress of the web material, ksi (MPa)
k = distance from outer face of the flange to the web toe of the fillet, in. (mm)
l_b = length of bearing, in. (mm)
t_w = thickness of web, in. (mm)

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

3. Web Local 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 $d/2$

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

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

(1) For $l_b/d \le 0.2$

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

(2) For $l_b/d > 0.2$

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

where
d = full nominal depth of the member, in. (mm)
Q_f = 1.0 for wide-flange sections and for HSS (connecting surface) in tension
= as given in Table K3.2 for all other HSS conditions

When required, a transverse stiffener, a pair of transverse stiffeners, or a doubler plate extending at least three quarters of 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 for the limit state of sidesway buckling shall be determined as follows:

$\phi = 0.85$ (LRFD) $\Omega = 1.76$ (ASD)

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

(a) If the compression flange is restrained against rotation

(1) When $(h/t_w)/(L_b/b_f) \le 2.3$

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

(2) When $(h/t_w)/(L_b/b_f) > 2.3$, 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

(1) When $(h/t_w)/(L_b/b_f) \le 1.7$

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

(2) When $(h/t_w)/(L_b/b_f) > 1.7$, 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:

C_r = 960,000 ksi (6.6×10^6 MPa), when $\alpha_s M_r < M_y$ at the location of the force
= 480,000 ksi (3.3×10^6 MPa), when $\alpha_s M_r < M_y$ at the location of the force

L_b = largest laterally unbraced length along either flange at the point of load, in. (mm)

M_r = required flexural strength using LRFD or ASD load combinations, kip-in. (N-mm)

b_f = width of flange, in. (mm)

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)

α_s = 1.0 (LRFD); 1.5 (ASD)

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 compression buckling shall be determined as follows:

$$R_n = \left(\frac{24 t_w^3 \sqrt{E F_{yw}}}{h} \right) Q_f \tag{J10-8}$$

ϕ = 0.90 (LRFD) Ω = 1.67 (ASD)

where

Q_f = 1.0 for wide-flange sections and for HSS (connecting surface) in tension.

= as given in Table K3.2 for all other HSS conditions

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%.

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:

ϕ = 0.90 (LRFD) Ω = 1.67 (ASD)

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

(a) When the effect of inelastic panel-zone deformation on frame stability is not accounted for in the analysis:

(1) For $\alpha P_r \le 0.4 P_y$

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

(2) For $\alpha P_r > 0.4 P_y$

$$R_n = 0.60 F_y d_c t_w \left(1.4 - \frac{\alpha P_r}{P_y}\right) \quad \text{(J10-10)}$$

(b) When frame stability, including plastic panel zone deformation, is considered in the analysis or when the required panel-zone shear strength is determined based on the plastic bending moment, M_p, flexural strength of the beam:

(1) For $\alpha P_r \le 0.75 P_y$

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

(2) For $\alpha P_r > 0.75 P_y$

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

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

A_g = gross area of member, in.^{2} (mm^2)
F_y = specified minimum yield stress of the column web, ksi (MPa)
P_r = required axial strength using LRFD or ASD load combinations, kips (N)
P_y = $F_y A_g$, axial yield strength of the column, kips (N)
b_{cf} = width of column flange, in. (mm)
d_b = depth of beam, in. (mm)
d_c = depth of column, in. (mm)
t_{cf} = thickness of column flange, in. (mm)
t_w = thickness of column web, in. (mm)
α = 1.0 (LRFD); = 1.6 (ASD)

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 Section J4.1 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 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 Section 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 Section E6.2 and Section 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 requirements:

(a) The width of each stiffener plus one-half the thickness of the column web shall not be less than one-third of the flange or moment connection plate width delivering the concentrated force.
(b) 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, nor less than the width divided by 16.
(c) Transverse stiffeners shall extend a minimum of one-half the depth of the member except as required in Sections J10.3, 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.

Doubler plates shall comply with the following additional requirements:

(a) The thickness and extent of the doubler plate shall provide the additional material necessary to equal or exceed the strength requirements.

(b) The doubler plate shall be welded to develop the proportion of the total force transmitted to the doubler plate.

10. Transverse Forces on Plate Elements

When a force is applied transverse to the plane of a plate element, the nominal strength shall consider the limit states of shear and flexure in accordance with Sections J4.2 and J4.5.

User Note: The flexural strength can be checked based on yield-line theory and the shear strength can be determined based on a punching shear model. See AISC *Steel Construction Manual* Part 9 for further discussion.

CHAPTER K

ADDITIONAL REQUIREMENTS FOR HSS AND BOX-SECTION CONNECTIONS

This chapter addresses additional requirements for connections to HSS members and box sections of uniform wall thickness, where seam welds between box-section elements are complete-joint-penetration (CJP) groove welds in the connection region. The requirements of Chapter J also apply.

The chapter is organized as follows:

K1. GENERAL PROVISIONS AND PARAMETERS FOR HSS CONNECTIONS

For the purposes of this chapter, the centerlines of branch members and chord members shall lie in a common plane. Rectangular HSS connections are further limited to having all members oriented with walls parallel to the plane.

The tables in this chapter are often accompanied by limits of applicability. Connections complying with the limits of applicability listed can be designed considering the limit states provided for each joint configuration. Connections not complying with the limits of applicability listed are not prohibited and must be designed by rational analysis.

User Note: The connection strengths calculated in Chapter K, including the applicable sections of Chapter J, are based on strength limit states only. See the Commentary if excessive connection deformations may cause serviceability or stability concerns.

User Note: Connection strength is often governed by the size of HSS members, especially the wall thickness of truss chords, and this must be considered in the initial design. To ensure economical and dependable connections can be designed, the connections should be considered in the design of the members. Angles between the chord and the branch(es) of less than 30° can make welding and inspection difficult and should be avoided. The limits of applicability provided reflect limitations on tests conducted to date, measures to eliminate undesirable limit states, and other considerations. See Section J3.10(c) for through-bolt provisions.

This section provides parameters to be used in the design of plate-to-HSS and HSS-to-HSS connections.

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

1. Definitions of Parameters

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

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

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

B_e = effective width of rectangular HSS branch member or plate for local yielding of the transverse element, in. (mm)

B_{ep} = effective width of rectangular HSS branch member or plate for punching shear, in. (mm)

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

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

F_c = available stress in main member, ksi (MPa)
= F_y for LRFD; $0.60F_y$ for ASD

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

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

F_{yb} = specified minimum yield stress of HSS branch member or plate material, 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)

Q_f = chord stress interaction parameter

l_{end} = distance from the near side of the connecting branch or plate to end of chord, in. (mm)

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

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

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

2. Rectangular HSS

2a. Effective Width for Connections to Rectangular HSS

For local yielding of transverse elements, the effective width of elements (plates or rectangular HSS branches) perpendicular to the longitudinal axis of a rectangular HSS member that deliver a force component transverse to the face of the member shall be taken as:

$$B_e = \left(\frac{10t}{B}\right)\left(\frac{F_y t}{F_{yb} t_b}\right) B_b \le B_b \tag{K1-1}$$

For shear yielding (punching), the effective width of the face of a rectangular HSS member, adjacent to transverse element (plates or rectangular HSS branches) shall be taken as:

$$B_{ep} = \left(\frac{10t}{B}\right) B_b \le B_b \tag{K1-2}$$

User Note: Section J4 addresses the strength of affected elements in tension, compression, flexure and shear. The effective widths above are used to establish the effective areas to be used when checking these limit states. The commentary provide further guidance

3. Main Member Stress Function

Where required, the main member stress function, Q_f, shall be taken as:

(a) For HSS connecting surface in tension, $Q_f = 1$

(b) For round HSS connecting surface in compression

$$Q_f = 1 - 0.3U\left(1+U\right) \le 1.0 \qquad \text{(K1-3)}$$

(c) For rectangular HSS connecting surface in compression

$$Q_f = 1.3 - 0.4\left(\frac{U}{\beta}\right) \le 1.0 \text{ for T-, Y-, and cross connections} \qquad \text{(K1-4)}$$

$$Q_f = 1.3 - 0.4\left(\frac{U}{\beta_{eff}}\right) \le 1.0 \text{ for gapped K-connections} \qquad \text{(K1-5)}$$

$$Q_f = \sqrt{1-U^2} \quad \text{for longitudinal plate connections} \qquad \text{(K1-6)}$$

where

$$U = \left|\frac{P_{ro}}{F_c A_g} + \frac{M_{ro}}{F_c S}\right| \le 1.0$$

where P_{ro} and M_{ro} are determined on the side of the joint that has lower compression stress for round HSS and higher compression stress for rectangular HSS. P_{ro} and M_{ro} refer to required strengths in the HSS: $P_{ro} = P_u$ for LRFD, and P_a for ASD; $M_{ro} = M_u$ for LRFD, and M_a for ASD.

Limits of Applicability:
$D/t \le 50$ for round HSS T-, Y-,and K-connections
$D/t \le 40$ for round HSS cross-connections
B/t and $H/t \le 35$ for rectangular HSS overlapped K-connections
$F_y \le 52$ ksi (360 MPa)
$F_y/F_u \le 0.8$ Note: ASTM A500 Grade C is acceptable

4. End Distance

The connection available strengths in Chapters J and K assume a main member with a minimum end distance, l_{end}, on both sides of a connections .

(a) For rectangular sections:

$$l_{end} \ge B\sqrt{1-B} \text{ , for } \beta \le 0.85 \qquad \text{(K1-7)}$$

(b) For round sections:

$$l_{end} \geq D\left(1.25 - \frac{\beta}{2}\right) \quad \text{(K1-8)}$$

When the connection occurs at a distance less than l_{end} from an open chord end, reduce the connection available strength by 50%.

K2. CONCENTRATED FORCES ON HSS

1. Definitions of Parameters

l_b = 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)

2. Round HSS

The available strength of plate-to-round HSS connections, within the limits in Table K2.1A, shall be taken as shown in Table K2.1.

TABLE K2.1
Available Strengths of Plate-to-Round HSS Connections

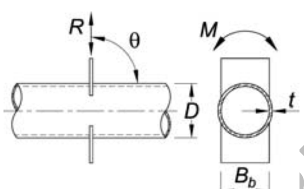
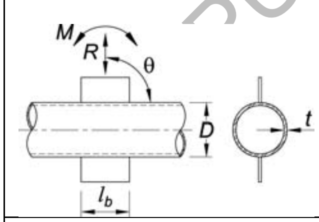
Connection Type	Connection Available Strength	Plate Bending	
Transverse Plate T-, Y-, and Cross-Connections	**Limit state: HSS local yielding**		
	Plate Axial Load	**In-Plane**	**Out-of-Plane**
	$R_n \sin\theta = F_y t^2 \left(\frac{5.5}{1-0.81\frac{B_b}{D}}\right) Q_f$ (K2-1a)	–	$M_n = 0.5 B_b R_n$ (K2-1b)
	$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)		
Longitudinal Plate T-, Y- and Cross-Connections	**Limit state: HSS plastification**		
	Plate Axial Load	**In-Plane**	**Out-of-Plane**
	$R_n \sin\theta = 5.5 F_y t^2 \left[1 + 0.25\frac{l_b}{D}\right] Q_f$ (K2-2a)	$M_n = 0.8\, l_b R_n$ (K2-2b)	–
	$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)		

TABLE K2.1A
Limits of Applicability of Table K2.1

HSS wall slenderness:	$D/t \le 50$ for T-connections under branch plate axial load or bending $D/t \le 40$ for cross-connections under branch plate axial load or bending
Width ratio:	$0.2 < B_b/D \le 1.0$ for transverse branch plate connections
Material strength:	$F_y \le 52$ ksi (360 MPa)
Ductility:	$F_y/F_u \le 0.8$ Note: ASTM A500 Gr. C is acceptable

3. Rectangular HSS

The available strength of connections to rectangular HSS with concentrated loads shall be determined based on the applicable limit states from Chapter J.

K3. 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 classified as a Y-connection otherwise.

(b) When the punching load, $P_r \sin\theta$, in a branch member is essentially equilibrated (within 20%) 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 adequacy of the connections shall be determined by interpolation on the proportion of the available strength of each in total.

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

O_v = $l_{ov}/l_p \times 100$, %

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

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

l_b = H_b / sinθ, in. (mm)

l_{ov} = overlap length measured along the connecting face of the chord beneath the two branches, in. (mm)

l_p = projected length of the overlapping branch on the chord, in. (mm)

γ = 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 = 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 = l_b/B

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

ζ = 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. Round HSS

The available strength of round HSS-to-HSS truss connections, within the limits in Table K3.1A, shall be taken as the lowest value obtained according to the limit states shown in Table K3.1.

TABLE K3.1
Available Strengths of Round HSS-to-HSS Truss Connections

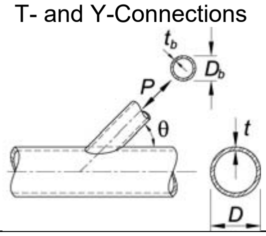
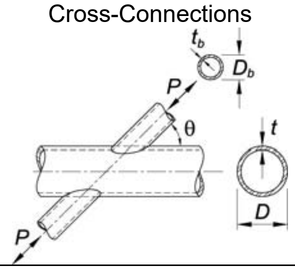
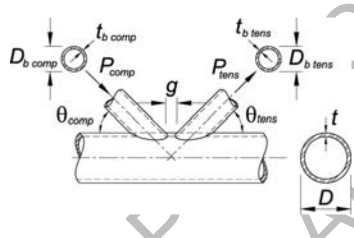
Connection Type	Connection Available Axial Strength
General Check for T-, Y-, Cross- and K-Connections with gap, when $D_{b\,(\text{tens/comp})} < (D - 2t)$	Limit State: Shear Yielding (punching) $P_n = 0.6 F_y t \pi D_b \left(\frac{1+\sin\theta}{2\sin^2\theta} \right)$ (K3-1) $\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)
T- and Y-Connections  	Limit State: Chord Plastification $P_n \sin\theta = F_y t^2 \left(3.1 + 15.6\beta^2\right) \gamma^{0.2} Q_f$ (K3-2) $\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)
Cross-Connections  	Limit State: Chord Plastification $P_n \sin\theta = F_y t^2 \left(\frac{5.7}{1 - 0.81\,\beta} \right) Q_f$ (K3-3) $\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)
K-Connections with Gap or Overlap  	Limit State: Chord Plastification $(P_n \sin\theta)_{compression\ branch}$ $= F_y t^2 \left(2.0 + 11.33 \frac{D_{b\,comp}}{D} \right) Q_g Q_f$ (K3-4) $(P_n \sin\theta)_{tension\ branch}$ $= (P_n \sin\theta)_{compression\ branch}$ (K3-5) $\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)
Functions	
$Q_g = \gamma^{0.2} \left[1 + \frac{0.024\gamma^{1.2}}{\exp\left(\frac{0.5g}{t} - 1.33 \right) + 1} \right]$ (K3-6) Note that exp(x) is equal to e^x, where e=2.71828 is the base of the natural logarithm.	

TABLE K3.1A
Limits of Applicability of Table K3.1

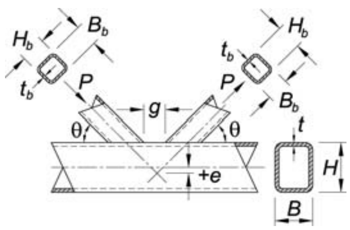
Joint eccentricity:	–0.55	$\le e/D \le$ 0.25 for K-connections
Chord wall slenderness:	D/t	$\le$ 50 for T-, Y- and K-connections
	D/t	$\le$ 40 for cross-connections
Branch wall slenderness:	D_b/t_b	$\le$ 50 for tension and compression branch
	D_b/t_b	$\le 0.05E/F_{yb}$ for compression branch
Width ratio:	0.2	$\le D_b/D \le 1.0$ for T-, Y-, cross- and overlapped K-connections
	0.4	$\le D_b/D \le 1.0$ for gapped K-connections
Gap:	g	$\le t_{b\ comp} + t_{b\ tens}$ for gapped K-connections
Overlap:	25%	$\le O_v \le$ 100% for overlapped K-connections
Branch thickness:	$t_{b\ overlapping}$	$\le t_{b\ overlapped}$ for branches in overlapped K-connections
Material strength:	F_y and F_{yb}	$\le$ 52 ksi (360 MPa)
Ductility strength:	F_y/F_u and F_{yb}/F_{ub}	$\le$ 0.8 Note: ASTM A500 Grade C is accpetable.

3. Rectangular HSS

The available strength, ϕP_n and P_n/Ω, of rectangular HSS-to-HSS truss connections within the limits in Table K3.2A, shall be taken as the lowest value obtained according to limit states shown in Table K3.2 and Chapter J.

User Note: Outside the limits in Table K3.2A, the limit states of Chapter J are still applicable and the applicable limit states of Chapter K are not defined.

<table>
<tr><th colspan="2">TABLE K3.2
Available Strengths of Rectangular
HSS-to-HSS Truss Connections</th></tr>
<tr><th>Connection Type</th><th>Connection Available Axial Strength</th></tr>
<tr><td rowspan="4">Gapped K-Connections



</td><td>Limit State: Chord Wall Plastification, for all β

$$P_n \sin\theta = F_y t^2 \left(9.8\beta_{eff}\gamma^{0.5}\right) Q_f \qquad \text{(K3-7)}$$

ϕ = 0.90 (LRFD) Ω = 1.67 (ASD)</td></tr>
<tr><td>Limit State: Shear Yielding (punching), when $B_b < B - 2t$

This limit state need not be checked for square branches.

$$P_n \sin\theta = 0.6 F_y t B \left(2\eta + \beta + \beta_{eop}\right) \qquad \text{(K3-8)}$$

ϕ = 0.95 (LRFD) Ω = 1.58 (ASD)</td></tr>
<tr><td>Limit State: Shear of Chord Side Walls in the Gap Region

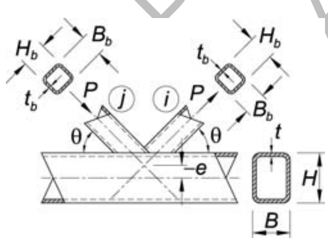
Determine $P_n \sin\theta$ in accordance with Section G4.

This limit state need not be checked for square chords.</td></tr>
<tr><td>Limit State: Local Yielding of Branch/Branches due to Uneven Load Distribution.

This limit state need not be checked for square branches or where $B/t \geq 15$.

$$P_n = F_{yb} t_b \left(2H_b + B_b + B_e - 4t_b\right) \qquad \text{(K3-9)}$$

ϕ = 0.95 (LRFD) Ω = 1.58 (ASD)</td></tr>
<tr><td>Overlapped K-Connections



Note that the force arrows shown for overlapped K-connections may be reversed; i and j control member identification.</td><td>Limit state: Local Yielding of Branch/Branches due to Uneven Load Distribution

ϕ = 0.95 (LRFD) Ω = 1.58 (ASD)

When 25% ≤ O_v < 50%

$$P_{n,i} = F_{ybi} t_{bi} \left[\frac{O_v}{50}\left(2H_{bi} - 4t_{bi}\right) + B_{ei} + B_{ej}\right] \qquad \text{(K3-10)}$$

When 50% ≤ O_v < 80%

$$P_{n,i} = F_{ybi} t_{bi} \left(2H_{bi} - 4t_{bi} + B_{ei} + B_{ej}\right) \qquad \text{(K3-11)}$$

When 80% ≤ O_v ≤ 100%

$$P_{n,i} = F_{ybi} t_{bi} \left(2H_{bi} - 4t_{bi} + B_{bi} + B_{ej}\right) \qquad \text{(K3-12)}$$

Subscript *i* refers to the overlapping branch

Subscript *j* refers to the overlapped branch

$$P_{n,j} = P_{n,i} \left(\frac{F_{ybj} A_{bj}}{F_{ybi} A_{bi}}\right) \qquad \text{(K3-13)}$$</td></tr>
</table>

Functions	
$\beta_{eff} = \left[(B_b + H_b)_{compression\ branch} + (B_b + H_b)_{tension\ branch} \right] / 4B$	(K3-16)
$\beta_{eop} = \dfrac{5\beta}{\gamma} \le \beta$	(K3-17)

TABLE K3.2A
Limits of Applicability of Table K3.2

Joint eccentricity:	–0.55	$\le e/H \le 0.25$ for K-connections
Chord wall slenderness:	B/t and H/t	≤ 35 for gapped K-connections and T-, Y-, and cross-connections
Branch wall slenderness:	B/t	≤ 30 for overlapped K-connections
	H/t	≤ 35 for overlapped K-connections
	B_b/t_b and H_b/t_b	≤ 35 for tension branch
		$\le 1.25\sqrt{\dfrac{E}{F_{yb}}}$ for compression branch of gapped K-, T-, Y- and cross-connections
		≤ 35 for compression branch of gapped K-, T-, Y- and cross-connections
		$\le 1.1\sqrt{\dfrac{E}{F_{yb}}}$ for compression branch of overlapped K-connections
Width ratio:	B_b/B and H_b/B	≥ 0.25 for T-, Y-, cross-, and overlapped K-connections
Aspect ratio:	0.5	$\le H_b/B_b \le 2.0$ and $0.5 \le H/B \le 2.0$
Overlap:	25%	$\le O_v \le 100\%$ for overlapped K-connections
Branch width ratio:	B_{bi}/B_{bj}	≥ 0.75 for overlapped K-connections, where subscript i refers to the overlapping branch and subscript j refers to the overlapped branch
Branch thickness ratio:	t_{bi}/t_{bj}	≤ 1.0 for overlapped K-connections, where subscript i refers to the overlapping branch and subscript j refers to the overlapped branch
Material strength:	F_y and F_{yb}	≤ 52 ksi (360 MPa)
Ductility:	F_y/F_u and F_{yb}/F_{ub}	≤ 0.8 Note: ASTM A500 Gr. C is acceptable.

Additional Limits for Gapped K-Connections		
Width ratio:	B_b/B and H_b/B	$\ge 0.1 + \dfrac{\gamma}{50}$
	β_{eff}	≥ 0.35
Gap ratio:	$\zeta = g/B$	$\ge 0.5(1 - \beta_{eff})$
Gap:	g	$\ge t_{b\ compression\ branch} + t_{b\ tension\ branch}$
Branch size:	smaller B_b	≥ 0.63 (larger B_b), if both branches are square

User Note: Maximum gap size in Table K3.2A will be controlled by the e/H limit. If the gap is large, treat as two Y-connections.

K4. 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 as:

(a) 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) A cross-connection when there is a branch on each (opposite) side of the chord

1. Definitions of Parameters

Z_b = Plastic section modulus of branch about the axis of bending, in.^{3} (mm^3)
β = width ratio
= D_b/D for round HSS; ratio of branch diameter to chord diameter
= B_b/B for rectangular HSS; ratio of overall branch width to chord width
γ = chord slenderness ratio
= $D/2t$ for round HSS; ratio of one-half the diameter to the wall thickness
= $B/2t$ for rectangular HSS; ratio of one-half the width to the wall thickness
η = load length parameter, applicable only to rectangular HSS
= l_b/B; the ratio of the length of contact of the branch with the chord in the plane of the connection to the chord width, where $l_b = H_b/\sin\theta$
θ = acute angle between the branch and chord (degrees)

2. Round HSS

The available strength of round HSS-to-HSS moment connections within the limits of Table K4.1A shall be taken as the lowest value of the applicable limit states shown in Table K4.1.

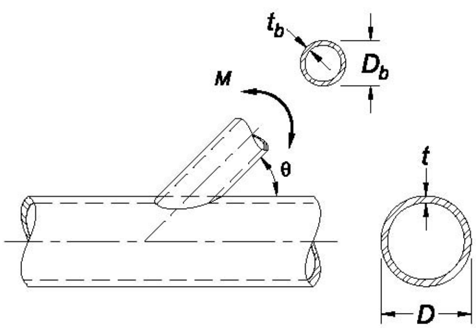
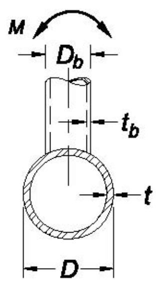
<table>
<tr><th colspan="2">TABLE K4.1
Available Strengths of Round HSS-to-HSS Moment Connections</th></tr>
<tr><th>Connection Type</th><th>Connection Available Flexural Strength</th></tr>
<tr><td rowspan="2">Branch(es) under In-Plane Bending
T-, Y- and Cross-Connections
</td><td>Limit State: Chord Plastification
$M_{n\text{-}ip} = 5.39F_y t^2 \gamma^{0.5} \beta \left(\frac{D_b}{\sin\theta}\right) Q_f$ (K4-1)
$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)</td></tr>
<tr><td>Limit State: Shear Yielding (punching), when $D_b < (D - 2t)$
$M_{n\text{-}ip} = 0.6F_y t D_b^2 \left(\frac{1+3\sin\theta}{4\sin^2\theta}\right)$ (K4-2)
$\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)</td></tr>
<tr><td rowspan="2">Branch(es) under Out-of-Plane Bending
T-, Y- and Cross-Connections
</td><td>Limit State: Chord Plastification
$M_{n\text{-}op} = \frac{F_y t^2 D_b}{\sin\theta}\left(\frac{3.0}{1-0.81\beta}\right) Q_f$ (K4-3)
$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)</td></tr>
<tr><td>Limit state: Shear Yielding (punching), when $D_b < (D - 2t)$
$M_{n\text{-}op} = 0.6F_y t D_b^2 \left(\frac{3+\sin\theta}{4\sin^2\theta}\right)$ (K4-4)
$\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)</td></tr>
<tr><td colspan="2">For T-, Y- and cross-connections, with branch(es) under combined axial load, in-plane bending, and out-of-plane bending, or any combination of these load effects:
$$\frac{P_r}{P_c} + \left(\frac{M_{r-ip}}{M_{c-ip}}\right)^2 + \frac{M_{r-op}}{M_{c-op}} \le 1.0 \quad \text{(K4-5)}$$
P_r = required axial strength in branch using LRFD or ASD load combinations, kips (N)
$M_{r\text{-}ip}$ = required in-plane flexural strength in branch using LRFD or ASD load combinations, kip-in (N-mm)
$M_{r\text{-}op}$= required out-of-plane flexural strength in branch using LRFD or ASD load combinations, kip-in (N-mm)
P_c = available axial strength obtained from Table K3.1, kips (N)
$M_{c\text{-}ip}$ = available strength for in-plane bending, kip-in (N-mm)
$M_{c\text{-}op}$= available strength for out-of-plane bending, kip-in (N-mm)</td></tr>
</table>

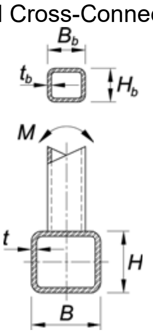
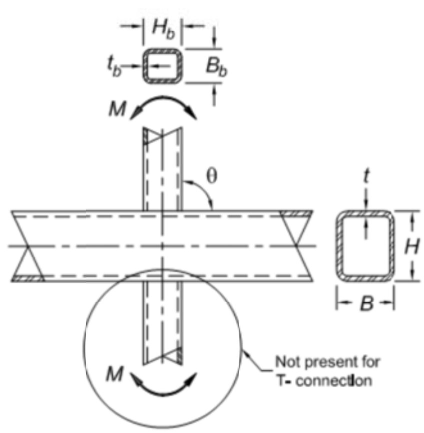
TABLE K4.1A
Limits of Applicability of Table K4.1

Chord wall slenderness:	$D/t \leq 50$ for T- and Y-connections
	$D/t \leq 40$ for cross-connections
Branch wall slenderness:	$D_b/t_b \leq 50$
	$D_b/t_b \leq 0.05E/F_{yb}$
Width ratio:	$0.2 < D_b/D \leq 1.0$
Material strength:	F_y and $F_{yb} \leq 52$ ksi (360 MPa)
Ductility:	F_y/F_u and $F_{yb}/F_{ub} \leq 0.8$ Note: ASTM A500 Gr. C is acceptable

3. Rectangular HSS

The available strength, ϕP_n and P_n/Ω, of rectangular HSS-to-HSS moment connections within the limits in Table K4.2A shall be taken as the lowest value obtained according to limit states shown in Table K4.2 and Chapter J.

User Note: Outside the limits in Table K4.2A, the limit states of Chapter J are still applicable and the applicable limit states of Chapter K are not defined.

<table>
<tr><th colspan="2">TABLE K4.2
Available Strengths of Rectangular HSS-to-HSS Moment Connections</th></tr>
<tr><th>Connection Type</th><th>Connection Available Flexural Strength</th></tr>
<tr><td rowspan="2">Branch(es) under Out-of-Plane Bending
T- and Cross-Connections

</td><td>Limit State: Chord Sidewall Local Yielding
$M_{n-op} = F_y^* t(B-t)(H_b+5t)$ (K4-6)
$\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)</td></tr>
<tr><td>Limit State: Chord distortional failure, for T-connections and unbalanced cross-connections
$M_{n-op} = 2F_y t\left[H_b t + \sqrt{BHt(B+H)}\right]$
(K4-7)
$\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)</td></tr>
<tr><td>Branch(es) under In-Plane Bending
T- and Cross-Connections

</td><td>Limit State: Sidewall Local Yielding
When $\beta \geq 0.85$
$M_{n-ip} = 0.5F_y^* t(H_b+5t)^2$ (K4-8)
$\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)</td></tr>
<tr><td colspan="2">For T- and cross-connections, with branch(es) under combined axial load, in-plane bending, and</td></tr>
</table>

out-of-plane bending, or any combination of these load effects:

$$\frac{P_r}{P_c}+\frac{M_{r-ip}}{M_{c-ip}}+\frac{M_{r-op}}{M_{c-op}}\leq 1.0 \qquad \text{(K4-9)}$$

P_r = required axial strength in branch using LRFD or ASD load combinations, kips (N)
$M_{r\text{-}ip}$ = required in-plane flexural strength in branch using LRFD or ASD load combinations, kip-in (N-mm)
$M_{r\text{-}op}$ = required out-of-plane flexural strength in branch using LRFD or ASD load combinations, kip-in (N-mm)
P_c = available axial strength obtained from Table K3.1, kips (N)
$M_{c\text{-}ip}$ = available strength for in-plane bending, kip-in (N-mm)

$M_{c\text{-}op}$ = available strength for out-of-plane bending, kip-in (N-mm)
= ϕM_{n-op} (LRFD); = M_{n-op}/Ω (ASD)

Functions

$F_y{}^*= F_y$ for T- connections and $0.8F_y$ for cross connections

$P_{ro} = P_u$ for LRFD, and P_a for ASD; $M_{ro} = M_u$ for LRFD, and M_a for ASD.

TABLE K4.2A
Limits of Applicability of Table K4.2

Branch angle:	$\theta \cong 90°$
Chord wall slenderness:	B/t and $H/t \leq 35$
Branch wall slenderness:	B_b/t_b and $H_b/t_b \leq 35$
	$\leq 1.25\sqrt{\frac{E}{F_{yb}}}$
Width ratio:	$B_b/B \geq 0.25$
Aspect ratio:	$0.5 \leq H_b/B_b \leq 2.0$ and $0.5 \leq H/B \leq 2.0$
Material strength:	F_y and $F_{yb} \leq 52$ ksi (360 MPa)
Ductility:	F_y/F_u and $F_{yb}/F_{ub} \leq 0.8$ Note: ASTM A500 Gr. C is acceptable

K5. WELDS OF PLATES AND BRANCHES TO HSS

The available strength of branch connections shall be determined considering the nonuniformity of load transfer along the line of weld, due to differences in relative stiffness of HSS walls in HSS-to-HSS connections and between elements in transverse plate-to-HSS connections, as follows:

$$R_n \text{ or } P_n = F_{nw} t_w l_e \qquad \text{(K5-1)}$$

$$M_{n\text{-}ip} = F_{nw} S_{ip} \qquad \text{(K5-2)}$$

$$M_{n\text{-}op} = F_{nw} S_{op} \qquad \text{(K5-3)}$$

Interaction shall be considered.

(a) For fillet welds

$\phi = 0.75$ (LRFD) $\quad \Omega = 2.00$ (ASD)

(b) For partial-joint-penetration groove welds

$\phi = 0.80$ (LRFD) $\Omega = 1.88$ (ASD)

where

F_{nw} = nominal stress of weld metal in accordance with Chapter J, ksi (MPa)

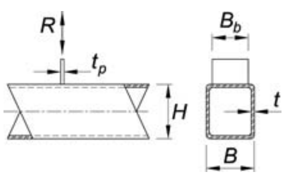
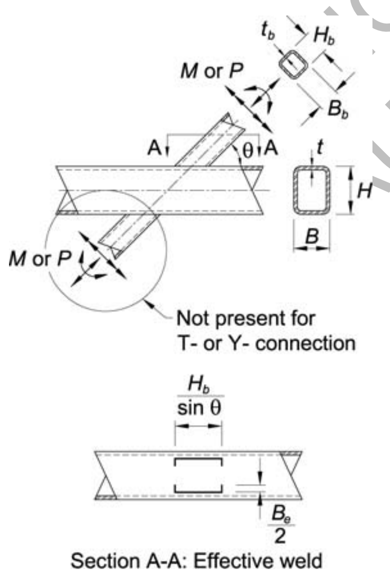
S_{ip} = effective elastic section modulus of welds for in-plane bending (Table K5.1), in.^{3} (mm^3)

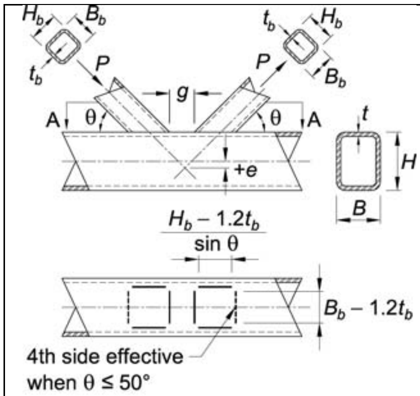
S_{op} = effective elastic section modulus of welds for out-of-plane bending (Table K5.1), in.^{3} (mm^3)

l_e = total effective weld length of groove and fillet welds to HSS for weld strength calculations, in. (mm)

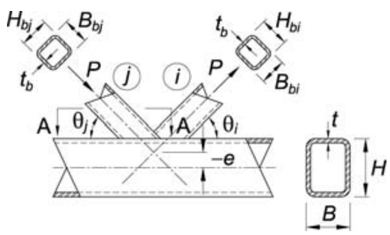
t_w = smallest effective weld throat around the perimeter of branch or plate, in. (mm)

TABLE K5.1
Effective Weld Properties for Connections to Rectangular HSS

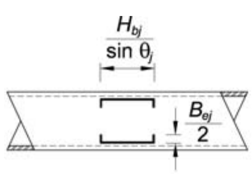
Connection Type	Weld Properties
Transverse Plate T- and Cross-Connections under Plate Axial Load 	Effective Weld Properties $l_e = 2B_e$ (K5-4) where l_e = total effective weld length for welds on both sides of the transverse plate
T-, Y-, and Cross-Connections under Branch Axial Load or Bending 	Effective Weld Properties $l_e = \frac{2H_b}{\sin\theta} + 2B_e$ (K5-5) $S_{ip} = \frac{t_w}{3}\left(\frac{H_b}{\sin\theta}\right)^2 + t_w B_e\left(\frac{H_b}{\sin\theta}\right)$ (K5-6) $S_{op} = t_w\left(\frac{H_b}{\sin\theta}\right)B_b + \frac{t_w}{3}\left(B_b^2\right) - \frac{(t_w/3)(B_b - B_e)^3}{B_b}$ (K5-7) When $\beta > 0.85$ or $\theta > 50°$, $B_e/2$ shall not exceed $B_b/4$
Gapped K-Connections under Branch Axial Load	Effective Weld Properties When $\theta \le 50°$:

<table>
<tr>
<td>


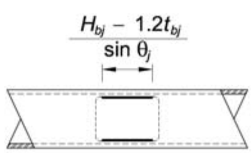
Section A-A: Effective weld for θ ≥ 60°
</td>
<td>
$$l_e = \frac{2(H_b - 1.2t_b)}{\sin\theta} + 2(B_b - 1.2t_b) \qquad \text{(K5-8)}$$
When $\theta \ge 60°$:
$$l_e = \frac{2(H_b - 1.2t_b)}{\sin\theta} + B_b - 1.2t_b \qquad \text{(K5-9)}$$
When $50° < \theta < 60°$, linear interpolation shall be used to determine l_e.
</td>
</tr>
<tr>
<td>
Overlapped K-Connections
Under Branch Axial Load



Note that the force arrows shown for overlapped K-connections may be reversed; i and j control member identification.



Section A-A: Effective weld when $\frac{B_{bj}}{B} \le 0.85$ and $\theta_j \le 50°$



Section A-A: Effective weld when $\frac{B_{bj}}{B} > 0.85$ or $\theta_j > 50°$
</td>
<td>
Overlapping Member Effective Weld Properties
(all dimensions are for the overlapping branch, i)

When $25\% \le O_v < 50\%$:
$$l_{e,i} = \frac{2O_v}{50}\left[\left(1-\frac{O_v}{100}\right)\left(\frac{H_{bi}}{\sin\theta_i}\right)+\frac{O_v}{100}\left(\frac{H_{bi}}{\sin(\theta_i+\theta_j)}\right)\right] + B_{ei} + B_{ej} \qquad \text{(K5-10)}$$

When $50\% \le O_v < 80\%$:
$$l_{e,i} = 2\left\{\left(1-\frac{O_v}{100}\right)\left(\frac{H_{bi}}{\sin\theta_i}\right)+\frac{O_v}{100}\left[\frac{H_{bi}}{\sin(\theta_i+\theta_j)}\right]\right\} + B_{ei} + B_{ej} \qquad \text{(K5-11)}$$

When $80\% \le O_v \le 100\%$:
$$l_{e,i} = 2\left[\left(1-\frac{O_v}{100}\right)\left(\frac{H_{bi}}{\sin\theta_i}\right)+\frac{O_v}{100}\left(\frac{H_{bi}}{\sin(\theta_i+\theta_j)}\right)\right] + B_{bi} + B_{ej} \qquad \text{(K5-12)}$$

When $B_{bi}/B > 0.85$ or $\theta_i > 50°$, $B_{ei}/2$ shall not exceed $B_{bi}/4$ and when $B_{bi}/B_{bj} > 0.85$ or $(180 - \theta_i - \theta_j) > 50°$, $B_{ej}/2$ shall not exceed $B_{bi}/4$.

Subscript i refers to the overlapping branch
Subscript j refers to the overlapped branch
<hr>
Overlapped Member Effective Weld Properties
(all dimensions are for the overlapped branch, j)
$$l_{e,j} = \frac{2H_{bj}}{\sin\theta_j} + 2B_{ej} \qquad \text{(K5-13)}$$
When $B_{bj}/B > 0.85$ or $\theta_j > 50°$,
$$l_{e,j} = 2(H_{bj} - 1.2t_{bj})/\sin\theta_j \qquad \text{(K5-14)}$$
</td>
</tr>
</table>

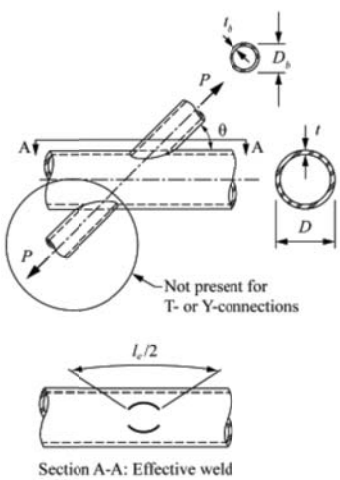
When a rectangular overlapped K-connection has been designed in accordance with Table K3.2, and the branch member component forces normal to the chord are 80%

balanced (i.e., the branch member forces normal to the chord face differ by no more than 20%), the hidden weld under an overlapping branch may be omitted if the remaining welds to the overlapped branch everywhere develop the full capacity of the overlapped branch member walls.

The weld checks in Tables K5.1 and K5.2 are not required if the welds are capable of developing the full strength of the branch member wall along its entire perimeter (or a plate along its entire length).

User Note: The approach used here to allow downsizing of welds assumes a constant weld size around the full perimeter of the HSS branch. Special attention is required for equal width (or near-equal width) connections to rectangular HSS which combine partial-joint-penetration groove welds along the matched edges of the connection, with fillet welds generally across the main member face.

TABLE K5.2
Effective Weld Properties for Connections to Round HSS

Connection Type	Weld Properties
T-, Y-, and Cross-Connections under Branch Axial Load 	Effective Weld Properties When 0.1≤ β ≤ 0.5, 60º ≤ θ ≤ 90º, and 10 ≤ D/t ≤ 50: $l_e = \frac{4}{\sqrt{2\beta(D/t)}} l_w \le l_w$ (K5-15) Where lwis the total weld length around the branch. This may be obtained from 3D models of 9ntersection cylinders, or from: $l_w = \pi D_b \frac{1+1/\sin\theta}{2}$ (K5-16)

CHAPTER L

DESIGN FOR SERVICEABILITY

This chapter addresses the evaluation of the structure and its components for the serviceability limit states of deflections, drift, vibration, wind-induced motion, thermal distortion, and connection slip.

The chapter is organized as follows:

L1. General Provisions
L2. Deflections
L3. Drift
L4. Vibration
L5. Wind-Induced Motion
L6. Thermal Expansion and Contraction
L7. Connection Slip

L1. GENERAL PROVISIONS

Serviceability is a state in which the function of a building, its appearance, maintainability, durability, and the comfort of its occupants are preserved under typical usage. Limiting values of structural behavior for serviceability (such as maximum deflections and accelerations) shall be chosen with due regard to the intended function of the structure. Serviceability shall be evaluated using applicable load combinations.

User Note: Serviceability limit states, service loads, and appropriate load combinations for serviceability considerations can be found in *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE/SEI 7) Appendix C and its commentary. The performance requirements for serviceability in this chapter are consistent with ASCE/SEI 7 Appendix C. Service loads are those that act on the structure at an arbitrary point in time and are not usually taken as the nominal loads.

Reduced stiffness values used in the direct analysis method, described in Chapter C, are not intended for use with the provisions of this chapter.

L2. DEFLECTIONS

Deflections in structural members and structural systems shall be limited so as not to impair the serviceability of the structure.

L3. DRIFT

Drift shall be limited so as not to impair the serviceability of the structure.

L4. 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 occupant loading, vibrating machinery and others identified for the structure.

L5. WIND-INDUCED MOTION

The effect of wind-induced motion of buildings on the comfort of occupants shall be considered.

L6. THERMAL EXPANSION AND CONTRACTION

The effects of thermal expansion and contraction of a building shall be considered.

L7. 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.

User Note: For the design of slip-critical connections, see Sections J3.8 and J3.9. For more information on connection slip, refer to the RCSC *Specification for Structural Joints Using High-Strength Bolts*.

CHAPTER M

FABRICATION AND ERECTION

This chapter addresses requirements for fabrication documents, fabrication, shop painting and erection.

The chapter is organized as follows:

M1. Fabriction and Erection Documents
M2. Fabrication
M3. Shop Painting
M4. Erection

M1. FABRICATION AND ERECTION DOCUMENTS

1. Fabrication Documents for Steel Construction

Fabrication documents shall indicate the work to be performed, and include items required by the applicable building code and the following as applicable:

(a) Locations of pretensioned bolts

(b) Locations of Class A, or higher, faying surfaces

(c) Weld access hole dimensions, surface profile and finish requirements

(d) Nondestructive testing (NDT) where performed by the fabricator

2. Erection Documents for Steel Construction

Erection documents shall indicate the work to be performed, and include items required by the applicable building code and the following as applicable:

(a) Locations of pretensioned bolts

(b) Those joints or groups of joints in which a specific assembly order, welding sequence, welding technique or other special precautions are required

User Note: Refer to *Code of Standard Practice* Section 4 addresses requirements for fabrication and erection documents.

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 regions shall not exceed 1,100°F (590°C) for ASTM A514/A514M nor 1,200°F (650°C) for other steels, unless limited by the specified ASTM material standard..

2. Thermal Cutting

Thermally cut edges shall meet the requirements of *Structural Welding Code—Steel* (AWS D1.1/D1.1M) clauses 5.14.5.2, 5.14.8.3 and 5.14.8.4, hereafter referred to as AWS D1.1M/D1.1M, with the exception that thermally cut free edges that will not be subject to fatigue 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 shall be formed with a curved transition. The radius need not exceed that required to fit the connection. Discontinuous corners are permitted where the material on both sides of the discontinuous reentrant corner are connected to a mating piece to prevent deformation and associated stress concentration at the corner.

User Note: Reentrant corners with a radius of 1/2 to 3/8 in. (13 to 10 mm) are acceptable for statically loaded work. Where pieces need to fit tightly together, a discontinuous reentrant corner is acceptable if the pieces are connected close to the corner on both sides of the discontinuous corner. Slots in HSS for gussets may be made with semicircular ends or with curved corners. Square ends are acceptable provided the edge of the gusset is welded to the HSS.

Weld access holes shall meet the geometrical requirements of Section J1.6. Beam copes and welds access holes in shapes that are to be galvanized shall be ground to bright metal. 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 *Surface Texture, Surface Roughness, Waviness, and Lay* (ASME B46.1), hereafter referred to as ASTM B46.1. 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.

User Note: The AWS *Surface Roughness Guide for Oxygen Cutting* (AWS C4.1-77) sample 2 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 construction documents or included in a stipulated edge preparation for welding.

4. Welded Construction

Welding shall be performed in accordance with AWS D1.1/D1.1M, except as modified in Section J2.

User Note: Welder qualification tests on plate defined in AWS D1.1/D1.1M clause 4 are appropriate for welds connecting plates, shapes or HSS to other plates, shapes or rectangular HSS. The 6GR tubular welder qualification is required for unbacked complete-joint-penetration groove welds of HSS T-, Y- and K-connections.

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 High-Strength Bolts* Section 3.3, hereafter referred to as the RCSC *Specification.* Water jet and thermally cut holes are permittedand shall have 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).

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 thermally cut holes.

Fully inserted finger shims, with a total thickness of not more than 1/4 in. (6 mm) within a joint, are permitted 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*, 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 equivalent means.

7. Dimensional Tolerances

Dimensional tolerances shall be in accordance with Chapter 6 of the AISC *Code of Standard Practice for Steel Buildings and Bridges*, hereafter referred to as the *Code of Standard Practice*.

8. Finish of Column Bases

Column bases and base plates shall be finished in accordance with the following requirements:

(a) Steel bearing plates 2 in. (50 mm) or less in thickness are permitted without milling provided a smooth and notch-free contact bearing surface 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 stipulated in subparagraphs (b) and (c) of this section, to obtain a smooth and notch-free contact bearing surface. Steel bearing plates over 4 in. (100 mm) in thickness shall be milled for bearing surfaces, except as stipulated in subparagraphs (b) and (c) of this section.

(b) Bottom surfaces of bearing plates and column bases that are grouted to ensure full bearing contact on foundations need not be milled.

(c) 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 are permitted to be mechanically or manually thermally cut, providing the quality requirements in accordance with the provisions of Section M2.2 are met.

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 otherwise protected from water infiltration.

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 buildup in enclosed parts.

User Note: Drainage and vent holes should be detailed on fabrication documents. See *The Design of Products to be Hot-Dip Galvanized After Fabrication*, American Galvanizer's Association, and ASTM A123, A143, A385, F2329, A385, and A780 for useful information on design and detailing of galvanized members. See Section M2.2 for requirements for copes of members that are to be galvanized.

M3. SHOP PAINTING

1. General Requirements

Shop painting and surface preparation shall be in accordance with the provisions in *Code of Standard Practice* Chapter 6.

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 construction documents.

3. Contact Surfaces

Paint is permitted in bearing-type connections. For slip-critical connections, the faying surface requirements shall be in accordance with RCSC *Specification* Section 3.2.2.

4. Finished Surfaces

Machine-finished surfaces shall be protected against corrosion by a rust inhibitive coating that can be removed prior to erection or 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 weld quality from meeting the quality requirements of this Specification, or produce unsafe fumes during welding.

M4. ERECTION

1. Column Base Setting

Column bases shall be set level and to correct elevation with full bearing on concrete or masonry as defined in *Code of Standard Practice* Section 7.

2. Stability and Connections

The frame of structural steel buildings shall be carried up true and plumb within the limits defined in *Code of Standard Practice* Chapter 7. As erection progresses, the structure shall be secured to support dead, erection and other loads anticipated to occur during the period of erection. Temporary bracing shall be provided, in accordance with the requirements of the *Code of Standard Practice*, 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 affected portions of the structure have been aligned as required by the construction documents.

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 equal to or 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 shims. Shims need not be other than mild steel, regardless of the grade of the main material.

5. Field Welding

Surfaces in and adjacent to joints to be field welded shall be prepared as necessary to assure weld quality. This preparation shall include surface preparation necessary to correct for damage or contamination occurring subsequent to fabrication.

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 contract documents.

CHAPTER N
QUALITY CONTROL AND QUALITY ASSURANCE

This chapter addresses minimum requirements for quality control, quality assurance and nondestructive testing for structural steel systems and steel elements of composite members for buildings and other structures.

User Note: This chapter does not address quality control or quality assurance for the following items:
(a) Steel (open web) joists and girders
(b) Tanks or pressure vessels
(c) Cables, cold-formed steel products, or gage material
(d) Concrete reinforcing bars, concrete materials, or placement of concrete for composite members
(e) Surface preparations or coatings

The Chapter is organized as follows:

N1. GENERAL PROVISIONS

Quality control (QC) and quality assurance (QA), as specified in this chapter, shall be provided. QC shall be provided by the fabricator and erector. QA shall be provided by others when required by the authority having jurisdiction (AHJ), applicable building code, purchaser, owner, engineer of record (EOR), or as modified by the provisions of N6. Nondestructive testing (NDT) shall be performed by the agency or firm responsible for quality assurance, except as permitted in accordance with Section N6.

User Note: The QA/QC requirements in Chapter N are considered adequate and effective for most steel structures and are strongly encouraged without modification. When the applicable building code and AHJ requires the use of a QA plan, this chapter outlines the minimum requirements deemed effective to provide satisfactory results in steel building construction. There may be cases where supplemental inspections are advisable. Additionally, where the contractor's QC program has demonstrated the capability to perform some tasks this plan has assigned to QA, modification of the plan could be considered.

User Note: The producers of materials manufactured in accordance with the standard specifications referenced in Section A3 and steel deck manufacturers are not considered to be fabricators or erectors.

N2. FABRICATOR AND ERECTOR QUALITY CONTROL PROGRAM

The fabricator and erector shall establish, maintain and implement QC procedures to ensure that their work is performed in accordance with this Specification and the construction documents.

1. Material Identification

Material identification procedures shall comply with the requirements of Section 6.1 of the AISC *Code of Standard Practice for Steel Buildings and Bridges*, hereafter referred to as the *Code of Standard Practice*, and shall be monitored by the fabricator's quality control inspector (QCI).

2. Fabricator Quality Control Procedures

The fabricator's QC procedures shall address inspection of the following as a minimum, as applicable:

(a) Shop welding, high-strength bolting, and details in accordance with Section N5
(b) Shop cut and finished surfaces in accordance with Section M2
(c) Shop heating for cambering, curving and straightening in accordance with Section M2.1
(d) Tolerances for shop fabrication in accordance with *Code of Standard Practice* Section 6.4

3. Erector Quality Control Procedures

The erector's quality control procedures shall address inspection of the following as a minimum, as applicable:

(a) Field welding, high-strength bolting, and details in accordance with Section N5
(b) Steel deck in accordance with SDI *Standard for Quality Control and Quality Assurance for Installation of Steel Deck*
(c) Headed steel stud anchor placement and attachment in accordance with Section N5.4
(d) Field cut surfaces in accordance with Section M2.2
(e) Field heating for straightening in accordance with Section M2.1
(f) Tolerances for field erection in accordance with *Code of Standard Practice* Section 7.13

N3. FABRICATOR AND ERECTOR DOCUMENTS

1. Submittals for Steel Construction

The fabricator or erector shall submit the following documents for review by the EOR or the EOR's designee, in accordance with *Code of Standard Practice* Section 4.4, prior to fabrication or erection, as applicable:

(a) Fabrication documents, unless fabrication documents have been furnished by others
(b) Erection documents, unless erection documents have been furnished by others

2. Available Documents for Steel Construction

The following documents shall be available in electronic or printed form for review by the EOR or the EOR's designee prior to fabrication or erection, as applicable, unless otherwise required in the construction documents to be submitted:

(a) For main structural steel elements, copies of material test reports in accordance with Section A3.1.
(b) For steel castings and forgings, copies of material test reports in accordance with Section A3.2.
(c) For fasteners, copies of manufacturer's certifications in accordance with Section A3.3.
(d) For anchor rods and threaded rods, copies of material test reports in accordance with Section A3.4.
(e) For welding consumables, copies of manufacturer's certifications in accordance with Section A3.5.
(f) For headed stud anchors, copies of manufacturer's certifications in accordance with Section A3.6.
(g) Manufacturer's product data sheets or catalog data for welding filler metals and fluxes to be used. The data sheets shall describe the product, limitations of use, recommended or typical welding parameters, and storage and exposure requirements, including baking, if applicable.
(h) Welding procedure specifications (WPS).
(i) Procedure qualification records (PQR) for WPS that are not prequalified in accordance with *Structural Welding Code—Steel* (AWS D1.1/D1.1M), hereafter referred to as AWS D1.1/D1.1M, or *Structural Welding Code—Sheet Steel* (AWS D1.3/D1.3M), as applicable.
(j) Welding personnel performance qualification records (WPQR) and continuity records.
(k) Fabricator's or erector's, as applicable, written QC manual that shall include, as a minimum:
    (1) Material control procedures
    (2) Inspection procedures
    (3) Nonconformance procedures
(l) Fabricator's or erector's, as applicable, QCI qualifications.
(m) Fabricator NDT personnel qualifications, if NDT is performed by the fabricator.

N4. INSPECTION AND NONDESTRUCTIVE TESTING PERSONNEL

1. Quality Control Inspector Qualifications

QC welding inspection personnel shall be qualified to the satisfaction of the fabricator's or erector's QC program, as applicable, and in accordance with either of the following:

(a) Associate welding inspectors (AWI) or higher as defined in *Standard for the Qualification of Welding Inspectors* (AWS B5.1), or
(b) Qualified under the provisions of AWS D1.1/D1.1M clause 6.1.4.

QC bolting inspection personnel shall be qualified on the basis of documented training and experience in structural bolting inspection.

The fabricator or erector's QCI performing coating inspection shall be qualified by training and experience as required by the firm's quality control program. The QCI shall receive initial and periodic documented training.

2. Quality Assurance Inspector Qualifications

QA welding inspectors shall be qualified to the satisfaction of the QA agency's written practice, and in accordance with either of the following:

(a) Welding inspectors (WI) or senior welding inspectors (SWI), as defined in *Standard for the Qualification of Welding Inspectors* (AWS B5.1), except AWI are permitted to be used under the direct supervision of WI, who are on the premises and available when weld inspection is being conducted, or
(b) Qualified under the provisions of AWS D1.1/D1.1M clause 6.1.4.

QA bolting inspection personnel shall be qualified on the basis of documented training and experience in structural bolting inspection.

QA coating inspection personnel shall be qualified to the satisfaction of the QA agency's written practice, receive documented training, have experience in coating inspection, and be qualified in accordance with one of the following:

(a) NACE, Coating Inspector Program (CIP) Level 1 Certification
(b) SSPC, Protective Coatings Inspector Program (PCI) Level 1 Certification
(c) On the basis of documented training and experience in coating application and inspection.

3. NDT Personnel Qualifications

NDT personnel, for NDT other than visual, shall be qualified in accordance with their employer's written practice, which shall meet or exceed the criteria of AWS D1.1/D1.1M clause 6.14.6, and,

(a) *Personnel Qualification and Certification Nondestructive Testing* (ASNT SNT-TC-1A), or
(b) *Standard for the Qualification and Certification of Nondestructive Testing Personnel* (ANSI/ASNT CP-189).

N5. MINIMUM REQUIREMENTS FOR INSPECTION OF STRUCTURAL STEEL BUILDINGS

1. Quality Control

QC inspection tasks shall be performed by the fabricator's or erector's QCI, as applicable, in accordance with Sections N5.4, N5.6 and N5.7.

Tasks in Tables N5.4-1 through N5.4-3 and Tables N5.6-1 through N5.6-3 listed for QC are those inspections performed by the QCI to ensure that the work is performed in accordance with the construction documents.

For QC inspection, the applicable construction documents are the fabrication documents and the erection documents, and the applicable referenced specifications, codes and standards.

User Note: The QCI need not refer to the design documents and project specifications. The *Code of Standard Practice* Section 4.2.1(a) requires the transfer of information from the contract documents (design documents and project specification) into accurate and complete fabrication and erection documents, allowing QC inspection to be based upon fabrication and erection documents alone.

2. Quality Assurance

The QAI shall review the material test reports and certifications as listed in Section N3.2 for compliance with the construction documents.

QA inspection tasks shall be performed by the QAI, in accordance with Sections N5.4, N5.6 and N5.7.

Tasks in Tables N5.4-1 through N5.4-3 and N5.6-1 through N5.6-3 listed for QA are those inspections performed by the QAI to ensure that the work is performed in accordance with the construction documents.

Concurrent with the submittal of such reports to the AHJ, EOR or owner, the QA agency shall submit to the fabricator and erector:

(a) Inspection reports
(b) NDT reports

3. Coordinated Inspection

When a task is noted to be performed by both QC and QA, it is permitted to coordinate the inspection function between the QCI and QAI so that the inspection functions are performed by only one party. When QA relies upon inspection functions performed by QC, the approval of the EOR and the AHJ is required.

4. Inspection of Welding

Observation of welding operations and visual inspection of in-process and completed welds shall be the primary method to confirm that the materials, procedures and workmanship are in conformance with the construction documents.

User Note: The technique, workmanship, appearance and quality of welded construction are addressed in Section M2.4.

As a minimum, welding inspection tasks shall be in accordance with Tables N5.4-1, N5.4-2 and N5.4-3. In these tables, the inspection tasks are as follows:

(a) Observe (O): The inspector shall observe these items on a random basis. Operations need not be delayed pending these inspections.
(b) Perform (P): These tasks shall be performed for each welded joint or member.

TABLE N5.4-1

Inspection Tasks Prior to Welding

Inspection Tasks Prior to Welding	QC	QA
Welder qualification records and continuity records	P	O
WPS available	P	P
Manufacturer certifications for welding consumables available	P	P
Material identification (type/grade)	O	O
Welder identification system[a]	O	O
Fit-up of groove welds (including joint geometry) • Joint preparations • Dimensions (alignment, root opening, root face, bevel) • Cleanliness (condition of steel surfaces) • Tacking (tack weld quality and location) • Backing type and fit (if applicable)	O	O
Fit-up of CJP groove welds of HSS T-, Y- and K-connections without backing (including joint geometry) • Joint preparations • Dimensions (alignment, root opening, root face, bevel) • Cleanliness (condition of steel surfaces) • Tacking (tack weld quality and location)	P	O
Configuration and finish of access holes	O	O
Fit-up of fillet welds • Dimensions (alignment, gaps at root) • Cleanliness (condition of steel surfaces) • Tacking (tack weld quality and location)	O	O
Check welding equipment	O	–

[a] The fabricator or erector, as applicable, shall maintain a system by which a welder who has welded a joint or member can be identified. Stamps, if used on cyclically-loaded members, require the approval of the engineer of record and shall be the low-stress type.

TABLE N5.4-2

Inspection Tasks During Welding

Inspection Tasks During Welding	QC	QA
Control and handling of welding consumables • Packaging • Exposure control	O	O
No welding over cracked tack welds	O	O
Environmental conditions • Wind speed within limits • Precipitation and temperature	O	O
WPS followed • Settings on welding equipment • Travel speed • Selected welding materials • Shielding gas type/flow rate • Preheat applied • Interpass temperature maintained (min./max.) • Proper position (F, V, H, OH)	O	O
Welding techniques • Interpass and final cleaning • Each pass within profile limitations • Each pass meets quality requirements	O	O
Placement and installation of steel headed stud anchors	P	P

TABLE N5.4-3

Inspection Tasks After Welding

Inspection Tasks After Welding	QC	QA
Welds cleaned	O	O
Size, length and location of welds	P	P
Welds meet visual acceptance criteria • Crack prohibition • Weld/base-metal fusion • Crater cross section • Weld profiles • Weld size • Undercut • Porosity	P	P
Arc strikes	P	P
k-area[a]	P	P
Weld access holes in rolled heavy shapes and built-up heavy shapes[b]	P	P
Backing removed and weld tabs removed (if required)	P	P
Repair activities	P	P
Document acceptance or rejection of welded joint or member [c]	P	P
No prohibited welds have been added without the approval of the EOR	O	O

[a] When welding of doubler plates, continuity plates or stiffeners has been performed in the *k*-area, visually inspect the web *k*-area for cracks within 3 in. (75 mm) of the weld.
[b] After rolled heavy shapes (see Section A3.1c) and built-up heavy shapes (see Section A3.1d) are welded, visually inspect the weld access hole for cracks.
[c] Stamps, if used on cyclically-loaded members, require the approval of the engineer of record and shall be the low-stress type.

5. Nondestructive Testing of Welded Joints

5a. Procedures

Ultrasonic testing (UT), magnetic particle testing (MT), penetrant testing (PT), and radiographic testing (RT), where required, shall be performed by QA in accordance with AWS D1.1/D1.1M.

User Note: The technique, workmanship, appearance and quality of welded construction is addressed in Section M2.4.

5b. CJP Groove Weld NDT

For structures in risk category III or IV, UT shall be performed by QA on all complete-joint-penetration (CJP) groove welds subject to transversely applied tension loading in butt, T- and corner joints, in material 5/16 in. (8 mm) thick or greater. For structures in risk category II, UT shall be performed by QA on 10% of CJP groove welds in butt, T- and corner joints subject to transversely applied tension loading, in materials 5/16 in. (8 mm) thick or greater.

User Note: For structures in risk category I, NDT of CJP groove welds is not required. For all structures in all risk categories, NDT of CJP groove welds in materials less than 5/16 in. (8 mm) thick is not required.

5c. Welded Joints Subjected to Fatigue

When required by Appendix 3, Table A-3.1, welded joints requiring weld soundness to be established by radiographic or ultrasonic inspection shall be tested by QA as prescribed. Reduction in the rate of UT is prohibited.

5d. Ultrasonic Testing Rejection Rate

The ultrasonic testing rejection rate shall be determined as the number of welds containing defects divided by the number of welds completed. Welds that contain acceptable discontinuities shall not be considered as having defects when the rejection rate is determined. For evaluating the rejection rate of continuous welds over 3 ft (1 m) in length where the effective throat is 1 in. (25 mm) or less, each 12 in. (300 mm) increment or fraction thereof shall be considered as one weld. For evaluating the rejection rate on continuous welds over 3 ft (1 m) in length where the effective throat is greater than 1 in. (25 mm), each 6 in. (150 mm) of length, or fraction thereof, shall be considered one weld.

5e. Reduction of Ultrasonic Testing Rate

For projects that contain 40 or fewer welds, there shall be no reduction in the ultrasonic testing rate. The rate of UT is permitted to be reduced if approved by the EOR and the AHJ. Where the initial rate of UT is 100%, the NDT rate for an individual welder or welding operator is permitted to be reduced to 25%, provided the rejection rate, the number of welds containing unacceptable defects divided by the number of welds completed, is demonstrated to be 5% or less of the welds tested for the welder or welding operator. A sampling of at least 40 completed welds shall be made for such reduced evaluation on each project.

5f. Increase in Ultrasonic Testing Rate

For structures in risk category II and higher (where the initial rate for UT is 10%) the NDT rate for an individual welder or welding operator shall be increased to 100% should the rejection rate (the number of welds containing unacceptable defects divided by the number of welds completed) exceed 5% of the welds tested for the welder or welding operator. A sampling of at least 20 completed welds on each project shall be made prior to implementing such an increase. If the rejection rate for the welder or welding operator falls to 5% or less on the basis of at least 40 completed welds, the rate of UT may be decreased to 10%.

5g. Documentation

All NDT performed shall be documented. For shop fabrication, the NDT report shall identify the tested weld by piece mark and location in the piece. For field work, the NDT report shall identify the tested weld by location in the structure, piece mark, and location in the piece.

When a weld is rejected on the basis of NDT, the NDT record shall indicate the location of the defect and the basis of rejection.

6. Inspection of High-Strength Bolting

Observation of bolting operations shall be the primary method used to confirm that the materials, procedures and workmanship incorporated in construction are in conformance with the construction documents and the provisions of the RCSC *Specification*.

(a) For snug-tight joints, pre-installation verification testing as specified in Table N5.6-1 and monitoring of the installation procedures as specified in Table N5.6-2 are not applicable. The QCI and QAI need not be present during the installation of fasteners in snug-tight joints.

(b) For pretensioned joints and slip-critical joints, when the installer is using the turn-of-nut or combined method with matchmarking techniques, the direct-tension-indicator method, or the twist-off-type tension control bolt method, monitoring of bolt pretensioning procedures shall be as specified in Table N5.6-2. The QCI and QAI need not be present during the installation of fasteners when these methods are used by the installer.

(c) For pretensioned joints and slip-critical joints, when the installer is using the turn-of-nut or combined method without matchmarking, or the calibrated wrench method, monitoring of bolt pretensioning procedures shall be as specified in Table N5.6-2. The QCI and QAI shall be engaged in their assigned inspection duties during installation of fasteners when these methods are used by the installer.

As a minimum, bolting inspection tasks shall be in accordance with Tables N5.6-1, N5.6-2 and N5.6-3. In these tables, the inspection tasks are as follows:

(a) Observe (O): The inspector shall observe these items on a random basis. Operations need not be delayed pending these inspections.
(b) Perform (P): These tasks shall be performed for each bolted connection.

TABLE N5.6-1

Inspection Tasks Prior to Bolting

Inspection Tasks Prior to Bolting	QC	QA
Manufacturer's certifications available for fastener materials	O	P
Fasteners marked in accordance with ASTM requirements	O	O
Correct fasteners selected for the joint detail (grade, type, bolt length if threads are to be excluded from shear plane)	O	O
Correct bolting procedure selected for joint detail	O	O
Connecting elements, including the appropriate faying surface condition and hole preparation, if specified, meet applicable requirements	O	O
Pre-installation verification testing by installation personnel observed and documented for fastener assemblies and methods used	P	O
Protected storage provided for bolts, nuts, washers and other fastener components	O	O

TABLE N5.6-2

Inspection Tasks During Bolting

Inspection Tasks During Bolting	QC	QA
Fastener assemblies placed in all holes and washers and nuts are positioned as required	O	O
Joint brought to the snug-tight condition prior to the pretensioning operation	O	O
Fastener component not turned by the wrench prevented from rotating	O	O
Fasteners are pretensioned in accordance with the RCSC *Specification*, progressing systematically from the most rigid point toward the free edges	O	O

TABLE N5.6-3

Inspection Tasks After Bolting

Inspection Tasks After Bolting	QC	QA
Document acceptance or rejection of bolted connections	P	P

7. Inspection of Galvanized Structural Steel Main Members

Exposed cut surfaces of galvanized structural steel main members and exposed corners of rectangular HSS shall be visually inspected for cracks subsequent to galvanizing. Cracks shall be repaired or the member shall be rejected.

User Note: It is normal practice for fabricated steel that requires hot dip galvanizing to be delivered to the galvanizer and then shipped to the jobsite. As a result, inspection on site is common.

8. Other Inspection Tasks

The fabricator's QCI shall inspect the fabricated steel to verify compliance with the details shown on the fabrication documents.

User Note: This includes such items as the correct application of shop joint details at each connection.

The erector's QCI shall inspect the erected steel frame to verify compliance with the field installed details shown on the erection documents.

User Note: This includes such items as braces, stiffeners, member locations, and correct application of field joint details at each connection.

The QAI shall be on the premises for inspection during the placement of anchor rods and other embedments supporting structural steel for compliance with the construction documents. As a minimum, the diameter, grade, type and length of the anchor rod or embedded item, and the extent or depth of embedment into the concrete, shall be verified and documented prior to placement of concrete.

The QAI shall inspect the fabricated steel or erected steel frame, as applicable, to verify compliance with the details shown on the construction documents.

User Note: This includes such items as braces, stiffeners, member locations and the correct application of joint details at each connection.

The acceptance or rejection of joint details and the correct application of joint details shall be documented.

N6. APPROVED FABRICATORS AND ERECTORS

When the fabricator or erector has been approved by the AHJ to perform all inspections without the involvement of a third-party, independent, QAI, the fabricator or erector shall perform and document all of the QA inspections required by this Chapter.
NDT of welds completed in an approved fabricator's shop is permitted to be performed by that fabricator when approved by the AHJ. When the fabricator performs the NDT, the NDT reports prepared by the fabricator's NDT personnel shall be available for review by the QA agency.

At completion of fabrication, the approved fabricator shall submit a certificate of compliance to the AHJ stating that the materials supplied and work performed by the fabricator are in accordance with the construction documents. At completion of erection, the approved erector shall submit a certificate of compliance to the AHJ stating that the materials supplied and work performed by the erector are in accordance with the construction documents.

N7. NONCONFORMING MATERIAL AND WORKMANSHIP

Identification and rejection of material or workmanship that is not in conformance with the construction documents is 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. Nonconforming material and workmanship shall be brought to the immediate attention of the fabricator or erector, as applicable.

Nonconforming material or workmanship shall be brought into conformance or made suitable for its intended purpose as determined by the EOR.

Concurrent with the submittal of such reports to the AHJ, EOR or owner, the QA agency shall submit to the fabricator and erector:

(a) Nonconformance reports
(b) Reports of repair, replacement or acceptance of nonconforming items

N8. MINIMUM REQUIREMENTS FOR SHOP OR FIELD APPLIED COATINGS

When coating or touch up is specified in the contract documents, the fabricator and/or erector shall establish, maintain, and implement QC procedures to ensure the proper application of coatings on structural steel in accordance with the manufacturer's product data sheet, unless there is direction to the contrary in the contract documents.

User Note: When there is a conflict between the manufacturer's product data sheet and the contract documents for the proper application of a coating, it is recommended to clarify with the engineer of record which will govern.

Observation of the coating process prior to, during, and after the application of the coating shall be the primary method to confirm that the coating material, procedures, and workmanship are in conformance with the manufacturer's product data sheet unless there is direction to the contrary in the contract documents.

APPENDIX 1

DESIGN BY ADVANCED ANALYSIS

This Appendix permits the use of advanced methods of structural analysis to directly model system and member imperfections, and/or allow for the redistribution of member and connection forces and moments as a result of localized yielding.

The appendix is organized as follows:

1.1. GENERAL REQUIREMENTS

The analysis methods permitted in this Appendix shall ensure that equilibrium and compatibility are satisfied for the structure in its deformed shape, including all flexural, shear, axial, and torsional deformations, and all other component and connection deformations that contribute to the displacements of the structure.

Design by the methods of this Appendix shall be conducted in accordance with Section B3.1, using load and resistance factor design (LRFD).

1.2. DESIGN BY ELASTIC ANALYSIS

1. General Stability Requirements

Design by a second-order elastic analysis that includes the direct modeling of system and member imperfections is permitted for all structures subject to the limitations defined in this section. All requirements of Section C1 apply, with additional requirements and exceptions as noted below. All load-dependent effects shall be calculated at a level of loading corresponding to LRFD load combinations.

The influence of torsion shall be considered, including its impact on member deformations and second-order effects.

The provisions of this method apply only to doubly symmetric members, including I-shapes, HSS and box sections, unless evidence is provided that the method is applicable to other member types.

2. Calculation of Required Strengths

For design using a second-order elastic analysis that includes the direct modeling of system and member imperfections, the required strengths of components of the structure shall be determined from an analysis conforming to Section C2, with additional requirements and exceptions as noted in the following.

2a. General Analysis Requirements

The analysis of the structure shall also conform to the following requirements:

(a) Torsional member deformations shall be considered in the analysis.

(b) The analysis shall consider geometric nonlinearities, including P-Δ, P-δ, and twisting effects as applicable to the structure. The use of the approximate procedures appearing in Appendix 8 is not permitted.

User Note: A rigorous second-order analysis of the structure is an important requirement for this method of design. Many analysis routines common in design offices are based on a more traditional second-order analysis approach that includes only P-Δ and P-δ effects without consideration of additional second-order effects related to member twist, which can be significant for some members with unbraced lengths near or exceeding L_r. The type of second-order analysis defined herein also includes the beneficial effects of additional member torsional strength and stiffness due to warping restraint, which can be conservatively neglected. Refer to the Commentary for additional information and guidance.

(c) In all cases, the analysis shall directly model the effects of initial imperfections due to both points of intersection of members displaced from their nominal locations (system imperfections), and initial out-of-straightness or offsets of members along their length (member imperfections). The magnitude of the initial displacements shall be the maximum amount considered in the design; the pattern of initial displacements shall be such that it provides the greatest destabilizing effect for the load combination being considered. The use of notional loads to represent either type of imperfection is not permitted.

User Note: Initial displacements similar in configuration to both displacements due to loading and anticipated buckling modes should be considered in the modeling of imperfections. The magnitude of the initial points of intersection of members displaced from their nominal locations (system imperfections) should be based on permissible construction tolerances, as specified in the AISC *Code of Standard Practice for Steel Buildings and Bridges* or other governing requirements, or on actual imperfections, if known. When these displacements are due to erection tolerances, 1/500 is often considered, based on the tolerance of the out-of-plumbness ratio specified in the *Code of Standard Practice*. For out-of-straightness of members (member imperfections), a 1/1000 out-of-straightness ratio is often considered. Refer to the Commentary for additional guidance.

2b. Adjustments to Stiffness

The analysis of the structure to determine the required strengths of components shall use reduced stiffnesses as defined in Section C2.3. Such stiffness reduction, including factors of 0.8 and τ_b, shall be applied to all stiffnesses that are considered to contribute to the stability of the structure. The use of notional loads to represent τ_b is not permitted.

User Note: Stiffness reduction should be applied to all member properties including torsional properties (GJ and EC_w) affecting twist of the member cross section. One practical method of including stiffness reduction is to reduce E and G by $0.8\tau_b$, thereby leaving all cross-section geometric properties at their nominal value.

Applying this stiffness reduction to some members and not others can, in some cases, result in artificial distortion of the structure under load and thereby lead to an unintended redistribution of forces. This can be avoided by applying the reduction to all members, including those that do not contribute to the stability of the structure.

3. Calculation of Available Strengths

For design using a second-order elastic analysis that includes the direct modeling of system and member imperfections, the available strengths of members and connections shall be calculated in accordance with the provisions of Chapters D through K, as applicable, except as defined below, with no further consideration of overall structure stability.

The nominal compressive strength of members, P_n, may be taken as the cross-section compressive strength, F_yA_g, or as F_yA_e for members with slender elements, where A_e is defined in Section E7.

1.3. DESIGN BY INELASTIC ANALYSIS

User Note: Design by the provisions of this section is independent of the requirements of Section 1.2.

1. General Requirements

The design strength of the structural system and its members and connections shall equal or exceed the required strength as determined by the inelastic analysis. The provisions of Section 1.3 do not apply to seismic design.

The inelastic analysis shall take into account: (a) flexural, shear, axial, and torsional member deformations, and all other component and connection deformations that contribute to the displacements of the structure; (b) second-order effects (including P-Δ, P-δ, and twisting effects); (c) geometric imperfections; (d) stiffness reductions due to inelasticity, including partial yielding of the cross section that may be accentuated by the presence of residual stresses; and (e) uncertainty in system, member, and connection strength and stiffness.

Strength limit states detected by an inelastic analysis that incorporates all of the preceding requirements in this Section are not subject to the corresponding provisions of this Specification when a comparable or higher level of reliability is provided by the analysis. Strength limit states not detected by the inelastic analysis shall be evaluated using the corresponding provisions of Chapters D through K.

Connections shall meet the requirements of Section B3.4.

Members and connections subject to inelastic deformations shall be shown to have ductility consistent with the intended behavior of the structural system. Force redistribution due to rupture of a member or connection is not permitted.

Any method that uses inelastic analysis to proportion members and connections to satisfy these general requirements is permitted. A design method based on inelastic analysis that meets the preceding strength requirements, the ductility requirements of Section 1.3.2, and the analysis requirements of Section 1.3.3 satisfies these general requirements.

2. Ductility Requirements

Members and connections with elements subject to yielding shall be proportioned such that all inelastic deformation demands are less than or equal to their inelastic deformation capacities. In lieu of explicitly ensuring that the inelastic deformation demands are less than or equal to their inelastic deformation capacities, the following requirements shall be satisfied for steel members subject to plastic hinging.

2a. Material

The specified minimum yield stress, F_y, of members subject to plastic hinging shall not exceed 65 ksi (450 MPa).

2b. Cross Section

The cross section of members at plastic hinge locations shall be doubly symmetric with width-to-thickness ratios of their compression elements not exceeding λ_{pd}, where λ_{pd} is equal to λ_p from Table B4.1b, except as modified below:

(a) For the width-to-thickness ratio, h/t_w, of webs of I-shaped members, rectangular HSS, and box sections subject to combined flexure and compression

(1) When $P_u/\phi_c P_y \le 0.125$

$$\lambda_{pd} = 3.76\sqrt{\frac{E}{F_y}}\left(1 - \frac{2.75P_u}{\phi_c P_y}\right) \tag{A-1-1}$$

(2) When $P_u/\phi_c P_y > 0.125$

$$\lambda_{pd} = 1.12\sqrt{\frac{E}{F_y}}\left(2.33 - \frac{P_u}{\phi_c P_y}\right) \ge 1.49\sqrt{\frac{E}{F_y}} \tag{A-1-2}$$

where

P_u = required axial strength in compression, using LRFD load combinations, kips (N)

P_y = $F_y A_g$ = axial yield strength, kips (N)

h = as defined in Section B4.1, in. (mm)

t_w = web thickness, in. (mm)

ϕ_c = resistance factor for compression = 0.90

(b) For the width-to-thickness ratio, b/t, of flanges of rectangular HSS and box sections, and for flange cover plates between lines of fasteners or welds

$$\lambda_{pd} = 0.94\sqrt{E/F_y} \quad \text{(A-1-3)}$$

where

b = as defined in Section B4.1, in. (mm)

t = as defined in Section B4.1, in. (mm)

(c) For the diameter-to-thickness ratio, D/t, of round HSS in flexure

$$\lambda_{pd} = 0.045\,E/F_y \quad \text{(A-1-4)}$$

where

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

2c. Unbraced Length

In prismatic member segments that contain plastic hinges, the laterally unbraced length, L_b, shall not exceed L_{pd}, determined as follows. For members subject to flexure only, or to flexure and axial tension, L_b shall be taken as the length between points braced against lateral displacement of the compression flange, or between points braced to prevent twist of the cross section. For members subject to flexure and axial compression, L_b shall be taken as the length between points braced against both lateral displacement in the minor axis direction and twist of the cross section.

(a) For I-shaped members bent about their major axis:

$$L_{pd} = \left(0.12 - 0.076\frac{M_1'}{M_2}\right)\frac{E}{F_y}r_y \quad \text{(A-1-5)}$$

where

r_y = radius of gyration about minor axis, in. (mm)

(1) When the magnitude of the bending moment at any location within the unbraced length exceeds M_2

$$M_1'/M_2 = +1 \quad \text{(A-1-6a)}$$

Otherwise:

(2) When $M_{mid} \le (M_1 + M_2)/2$

$$M_1' = M_1 \quad \text{(A-1-6b)}$$

(3) When $M_{mid} > (M_1 + M_2)/2$

$$M_1' = (2M_{mid} - M_2) < M_2 \quad \text{(A-1-6c)}$$

where

M_1 = smaller moment at end of unbraced length, kip-in. (N-mm)

M_2 = larger moment at end of unbraced length, kip-in. (N-mm) (shall be taken as positive in all cases)

M_{mid} = moment at middle of unbraced length, kip-in. (N-mm)

M_1' = effective moment at end of unbraced length opposite from M_2, kip-in. (N-mm)

The moments M_1 and M_{mid} are individually taken as positive when they cause compression in the same flange as the moment, M_2, and taken as negative otherwise.

(b) For solid rectangular bars and for rectangular HSS and box sections bent about their major axis

$$L_{pd} = \left(0.17 - 0.10\frac{M_1'}{M_2}\right)\frac{E}{F_y}r_y \geq 0.10\frac{E}{F_y}r_y \qquad \text{(A-1-7)}$$

For all types of members subject to axial compression and containing plastic hinges, the laterally unbraced lengths about the cross section major and minor axes shall not exceed $4.71r_x\sqrt{E/F_y}$ and $4.71r_y\sqrt{E/F_y}$, respectively.

There is no L_{pd} limit for member segments containing plastic hinges in the following cases:

(a) Members with round or square cross sections subject only to flexure or to combined flexure and tension
(b) Members subject only to flexure about their minor axis or combined tension and flexure about their minor axis
(c) Members subject only to tension

2d. Axial Force

To ensure ductility in compression members with plastic hinges, the design strength in compression shall not exceed $0.75F_yA_g$.

3. Analysis Requirements

The structural analysis shall satisfy the general requirements of Section 1.3.1. These requirements are permitted to be satisfied by a second-order inelastic analysis meeting the requirements of this Section.

Exception: For continuous beams not subject to axial compression, a first-order inelastic or plastic analysis is permitted and the requirements of Sections 1.3.3b and 1.3.3c are waived.

User Note: Refer to the Commentary for guidance in conducting a traditional plastic analysis and design in conformance with these provisions.

3a. Material Properties and Yield Criteria

The specified minimum yield stress, F_y, and the stiffness of all steel members and connections shall be reduced by a factor of 0.9 for the analysis, except as stipulated in Section 1.3.3c.

The influence of axial force, major axis bending moment, and minor axis bending moment shall be included in the calculation of the inelastic response.

The plastic strength of the member cross section shall be represented in the analysis either by an elastic-perfectly-plastic yield criterion expressed in terms of the axial force, major axis bending moment, and minor axis bending moment, or by explicit modeling of the material stress-strain response as elastic-perfectly-plastic.

3b. Geometric Imperfections

In all cases, the analysis shall directly model the effects of initial imperfections due to both points of intersection of members displaced from their nominal locations (system imperfections), and initial out-of-straightness or offsets of members along their length (member imperfections). The magnitude of the initial displacements shall be the maximum amount considered in the design; the pattern of initial displacements shall be such that it provides the greatest destabilizing effect.

3c. Residual Stress and Partial Yielding Effects

The analysis shall include the influence of residual stresses and partial yielding. This shall be done by explicitly modeling these effects in the analysis or by reducing the stiffness of all structural components as specified in Section C2.3.

If the provisions of Section C2.3 are used, then:

(a) The 0.9 stiffness reduction factor specified in Section 1.3.3a shall be replaced by the reduction of the elastic modulus, E, by 0.8 as specified in Section C2.3, and

(b) the elastic-perfectly-plastic yield criterion, expressed in terms of the axial force, major axis bending moment, and minor axis bending moment, shall satisfy the cross-section strength limit defined by Equations H1-1a and H1-1b using $P_c = 0.9P_y$, $M_{cx} = 0.9M_{px}$, and $M_{cy} = 0.9M_{py}$.

~~APPENDIX 2~~
~~DESIGN FOR PONDING~~

Comment [DC1]: This Appendix is being proposed to be removed.

~~This appendix provides methods for determining whether a roof system has adequate strength and stiffness to resist ponding. These methods are valid for flat roofs with rectangular bays where the beams are uniformly spaced and the girders are considered to be uniformly loaded.~~

~~The appendix is organized as follows:~~

~~2.1. Simplified Design for Ponding~~
~~2.2. Improved Design for Ponding~~

~~The members of a roof system shall be considered to have adequate strength and stiffness against ponding by satisfying the requirements of Sections 2.1 or 2.2.~~

~~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:~~

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$$\cancel{C_p + 0.9C_s \le 0.25} \quad \text{(A-2-1)}$$

$$\cancel{I_d \ge 25\left(S^4\right)10^{-6}} \quad \text{(A-2-2)}$$

$$\cancel{I_d \ge 3940S^4} \quad \text{(A-2-2M)}$$

~~where~~

$$\cancel{C_p = \frac{32L_sL_p^4}{10^7 I_p}} \quad \text{(A-2-3)}$$

$$\cancel{C_p = \frac{504L_sL_p^4}{I_p}} \quad \text{(A-2-3M)}$$

$$\cancel{C_s = \frac{32SL_s^4}{10^7 I_s}} \quad \text{(A-2-4)}$$

$$\cancel{C_s = \frac{504SL_s^4}{I_s}} \quad \text{(A-2-4M)}$$

~~I_d = moment of inertia of the steel deck supported on secondary members, in.^{4} per ft (mm^4 per 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)~~
~~L_p = length of primary members, ft (m)~~
~~L_s = length of secondary members, ft (m)~~
~~S = spacing of secondary members, ft (m)~~

~~For trusses and steel joists, the calculation of the moments of inertia, I_p and I_s, shall include the effects of web member strain when used in the above equation.~~

~~**User Note**: When the moment of inertia is calculated using only the truss or joist chord areas, the reduction in the moment of inertia due to web member strain can typically be taken as 15%.~~

~~A steel deck shall be considered a secondary member when it is directly supported by the primary members.~~

~~2.2. IMPROVED DESIGN FOR PONDING~~

~~It is permitted to use the provisions in this section when a more accurate evaluation of framing stiffness is needed than that given by Equations A-2-1 and A-2-2.~~

~~Define the stress indexes~~

$$U_p = \left(\frac{0.8F_y - f_o}{f_o}\right)_p \quad \text{for the primary member} \tag{A-2-5}$$

$$U_s = \left(\frac{0.8F_y - f_o}{f_o}\right)_s \quad \text{for the secondary member} \tag{A-2-6}$$

~~where~~

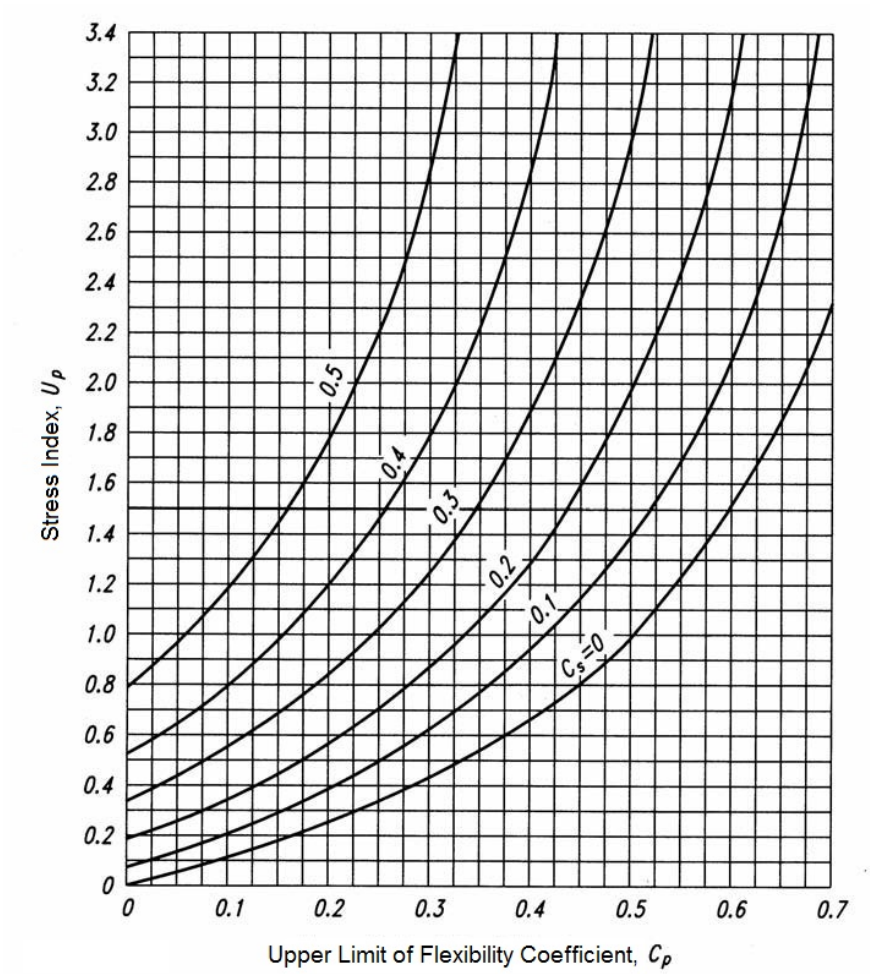
~~F_y = specified minimum yield stress, ksi (MPa)~~

~~f_o = stress due to impounded water due to either nominal rain or snow loads (exclusive of the ponding contribution), and other loads acting concurrently as specified in Section B2, ksi (MPa)~~

~~For roof framing consisting of primary and secondary members, evaluate the combined stiffness 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 coefficient 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.~~

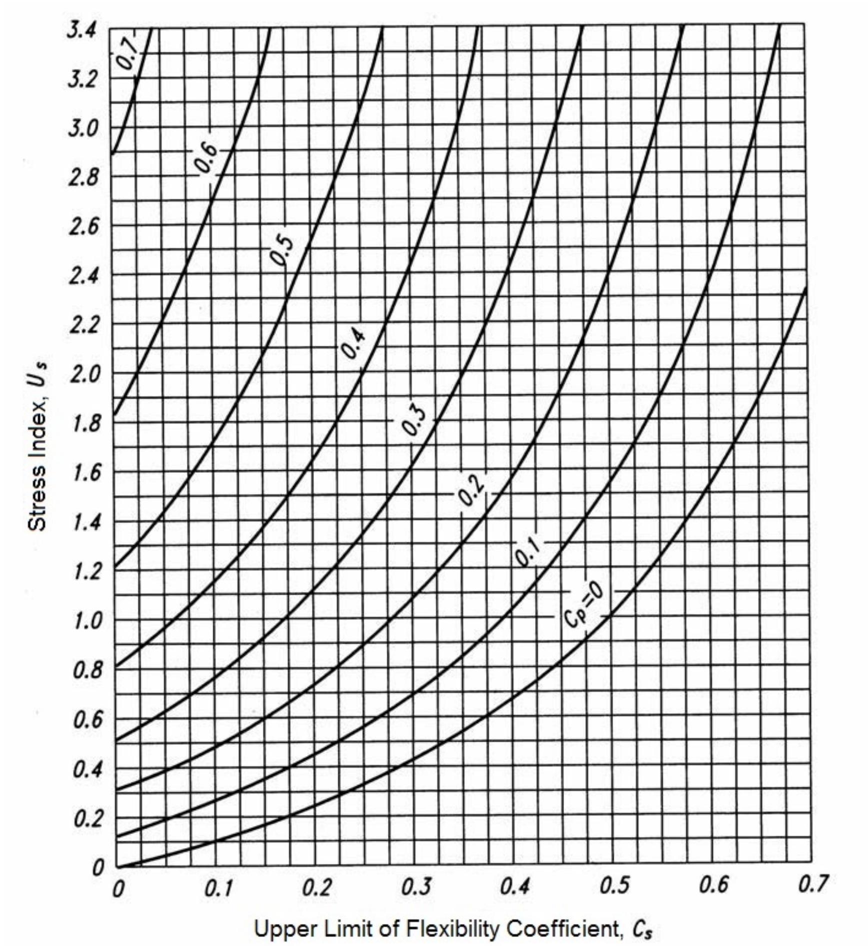
~~A similar procedure must be followed using Figure A-2.2.~~

~~For roof framing consisting of a series of equally spaced wall bearing beams, evaluate the stiffness 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 $C_p = 0$.~~



~~*Fig. A-2.1. Limiting flexibility coefficient for the primary systems.*~~

~~Evaluate the stability against ponding of a roof consisting of a metal roof deck of relatively slender depth-to-span ratio, spanning between beams supported directly on columns, as follows. Use Figure A-2.1 or A-2.2, using as C_s the flexibility coefficient for a one-foot (one-meter) width of the roof deck (S = 1.0).~~



~~Fig.A-2.2. Limiting flexibility coefficient for the secondary systems.~~

APPENDIX 3

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.

User Note: See AISC *Seismic Provisions for Structural Steel Buildings* for structures subject to seismic loads.

The appendix is organized as follows:

3.1. GENERAL PROVISIONS

The fatigue resistance of members consisting of shapes or plate shall be determined when the number of cycles of application of live load exceeds 20,000. No evaluation of fatigue resistance of members consisting of HSS in building-type structures subject to code mandated wind loads is required. When the applied cyclic stress range is less than the threshold allowable stress range, F_{TH}, no further evaluation of fatigue resistance is required. See Table A-3.1.

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.

The provisions of this Appendix shall apply to stresses calculated on the basis of the applied cyclic load spectrum. The maximum permitted stress due to peak cyclic loads shall be $0.66F_y$. 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.

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).

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. PLAIN MATERIAL AND WELDED JOINTS

In plain material and welded joints, the range of stress due to the applied cyclic loads shall not exceed the allowable stress range computed as follows.

(a) For stress categories A, B, B′, C, D, E and E,′ the allowable stress range, F_{SR}, shall be determined by Equation A-3-1 or A-3-1M, as follows:

$$F_{SR} = 1{,}000\left(\frac{C_f}{n_{SR}}\right)^{0.333} \geq F_{TH} \tag{A-3-1}$$

$$F_{SR} = 6\,900\left(\frac{C_f}{n_{SR}}\right)^{0.333} \geq F_{TH} \tag{A-3-1M}$$

where

C_f = constant from Table A-3.1 for the fatigue category

F_{SR} = allowable stress range, ksi (MPa)

F_{TH} = threshold allowable stress range, maximum stress range for indefinite design life from Table A-3.1, ksi (MPa)

n_{SR} = number of stress range fluctuations in design life

(b) For stress category F, the allowable stress range, F_{SR}, shall be determined by Equation A-3-2 or A-3-2M as follows:

$$F_{SR} = 100\left(\frac{1.5}{n_{SR}}\right)^{0.167} \geq 8 \text{ ksi} \tag{A-3-2}$$

$$F_{SR} = 690\left(\frac{1.5}{n_{SR}}\right)^{0.167} \geq 55 \text{ MPa} \tag{A-3-2M}$$

(c) For tension-loaded plate elements connected at their end by cruciform, T or corner details with partial-joint-penetration (PJP) groove welds transverse to the direction of stress, with or without reinforcing or contouring fillet welds, or if joined with only fillet welds, the allowable stress range on the cross section of the tension-loaded plate element shall be determined as the lesser of the following:

(1) Based upon crack initiation from the toe of the weld on the tension-loaded plate element (i.e., when R_{PJP} = 1.0), the allowable stress range, F_{SR}, shall be determined by Equation A-3-1 or A-3-1M for stress category C.

(2) Based upon crack initiation from the root of the weld, the allowable stress range, F_{SR}, on the tension loaded plate element using transverse PJP groove welds, with or without reinforcing or contouring fillet welds, the allowable stress range on the cross section at the root of the weld shall be determined by Equation A-3-3 or A-3-3M, for stress category C′ as follows:

$$F_{SR} = 1{,}000 R_{PJP}\left(\frac{4.4}{n_{SR}}\right)^{0.333} \tag{A-3-3}$$

$$F_{SR} = 6900 R_{PJP}\left(\frac{4.4}{n_{SR}}\right)^{0.333} \tag{A-3-3M}$$

where

R_{PJP}, the reduction factor for reinforced or nonreinforced transverse PJP groove welds, is determined as follows:

$$R_{PJP} = \frac{0.65 - 0.59\left(\dfrac{2a}{t_p}\right) + 0.72\left(\dfrac{w}{t_p}\right)}{t_p^{0.167}} \leq 1.0 \tag{A-3-4}$$

$$R_{PJP} = \frac{1.12 - 1.01\left(\dfrac{2a}{t_p}\right) + 1.24\left(\dfrac{w}{t_p}\right)}{t_p^{0.167}} \leq 1.0 \tag{A-3-4M}$$

$2a$ = length of the nonwelded root face in the direction of the thickness of the tension-loaded plate, in. (mm)

t_p = thickness of tension loaded plate, in. (mm)

w = leg size of the reinforcing or contouring fillet, if any, in the direction of the thickness of the tension-loaded plate, in. (mm)

If R_{PJP} = 1.0, the stress range will be limited by the weld toe and category C will control.

(3) Based upon crack initiation from the roots of a pair of transverse fillet welds on opposite sides of the tension loaded plate element, the allowable stress range, F_{SR}, on the cross section at the root of the welds shall be determined by Equation A-3-5 or A-3-5M, for stress category C″ as follows:

$$F_{SR} = 1{,}000 R_{FIL} \left(\frac{4.4}{n_{SR}} \right)^{0.333} \tag{A-3-5}$$

$$F_{SR} = 6900 R_{FIL} \left(\frac{4.4}{n_{SR}} \right)^{0.333} \tag{A-3-5M}$$

where

R_{FIL} = reduction factor for joints using a pair of transverse fillet welds only

$$= \frac{0.06 + 0.72\left(w / t_p\right)}{t_p^{0.167}} \le 1.0 \tag{A-3-6}$$

$$= \frac{0.103 + 1.24\left(w / t_p\right)}{t_p^{0.167}} \le 1.0 \tag{A-3-6M}$$

If R_{FIL} = 1.0, the stress range will be limited by the weld toe and category C will control.

User Note: Stress categories C′ and C″ are cases where the fatigue crack initiates in the root of the weld. These cases do not have a fatigue threshold and cannot be designed for an infinite life. Infinite life can be approximated by use of a very high cycle life such as 2×10^8. Alternatively, if the size of the weld is increased such that R_{FIL} or R_{PJP} is equal to 1.0, then the base metal controls, resulting in stress category C, where there is a fatigue threshold and the crack initiates at the toe of the weld.

3.4. BOLTS AND THREADED PARTS

In bolts and threaded parts, the range of stress of the applied cyclic load shall not exceed the allowable stress range computed as follows.

(a) For mechanically fastened connections loaded in shear, the maximum range of stress in the connected material of the applied cyclic load shall not exceed the allowable stress range computed using Equation A-3-1 or A-3-1M, where C_f and F_{TH} are taken from Section 2 of Table A-3.1.

(b) For high-strength bolts, common bolts, threaded anchor rods, and hanger 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 allowable stress range computed using Equation A-3-1 or A-3-1M, where C_f and F_{TH} are taken from Case 8.5 (stress category G). The net area in tension, A_t, is given by Equation A-3-7 or A-3-7M.

$$A_t = \frac{\pi}{4}\left(d_b - \frac{0.9743}{n}\right)^2 \quad \text{(A-3-7)}$$

$$A_t = \frac{\pi}{4}\left(d_b - 0.9382\,p\right)^2 \quad \text{(A-3-7M)}$$

where

d_b = nominal diameter (body or shank diameter), in. (mm)
n = threads per in. (per mm)
p = pitch, in. per thread (mm per thread)

For joints in which the material within the grip is not limited to steel or joints that 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 pretensioned to the requirements of Table J3.1 or J3.1M, an analysis of the relative stiffness of the connected parts and bolts is permitted to be used to determine the tensile stress range in the pretensioned bolts due to the total applied cyclic 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% of the absolute value of the applied cyclic axial load and moment from dead, live and other loads.

The following exceptions to RCSC Specifications shall apply to cyclically loaded connections subject to the requirements of Appendix 3:

(a) The Engineer’s approval for thermally cut holes is not required for cyclically loaded slip critical joints.
(b) Both pretensioned and not pretensioned bolted connections are permitted.
(c) Bolt net tension area is used for tension
(d) The maximum allowable stress range and stress range threshold are independent of the bolt material.

3.5. FABRICATION AND ERECTION REQUIREMENTS FOR FATIGUE

Longitudinal steel backing, if used, shall be continuous. If splicing of steel backing is required for long joints, the splice shall be made with a complete-joint-penetration (CJP) groove weld, ground flush to permit a tight fit. If fillet welds are used to attach left-in-place longitudinal backing, they shall be continuous.

In transverse CJP groove welded T- and corner-joints, a reinforcing fillet weld, not less than 1/4 in. (6 mm) in size, shall be added at reentrant corners.

The surface roughness of thermally cut edges subject to cyclic stress ranges, that include tension, shall not exceed 1,000 μin. (25 μm), where *Surface Texture, Surface Roughness, Waviness, and Lay* (ASME B46.1) is the reference standard.

User Note: AWS C4.1 Sample 3 may be used to evaluate compliance with this requirement.

Reentrant corners at cuts, copes and weld access holes shall form a radius not less than the prescribed radius in Table A-3.1.

For transverse butt joints in regions of tensile stress, weld tabs shall be used to provide for cascading the weld termination outside the finished joint. End dams shall not be used. Weld tabs shall be removed and the end of the weld finished flush with the edge of the member.

Fillet welds subject to cyclic loading normal to the outstanding legs of angles or on the outer edges of end plates shall have end returns around the corner for a distance not less than two times the weld size; the end return distance shall not exceed four times the weld size.

3.6. NONDESTRUCTIVE EXAMINATION REQUIREMENTS FOR FATIGUE

In the case of CJP groove welds, the maximum allowable stress range calculated by Equation A-3-1 or A-3-1M applies only to welds that have been ultrasonically or radiographically tested and meet the acceptance requirements of *Structural Welding Code—Steel* (AWS D1.1/D1.1M) clause 6.12.2 or clause 6.13.2.

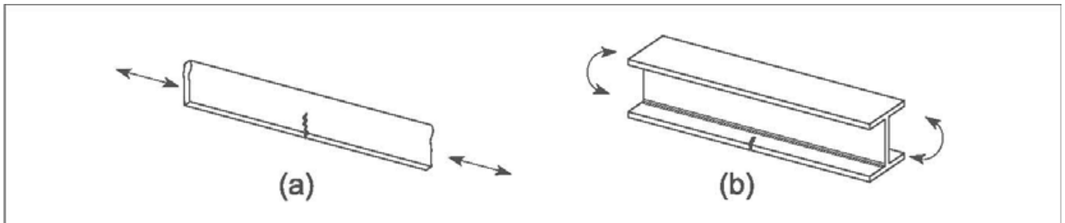
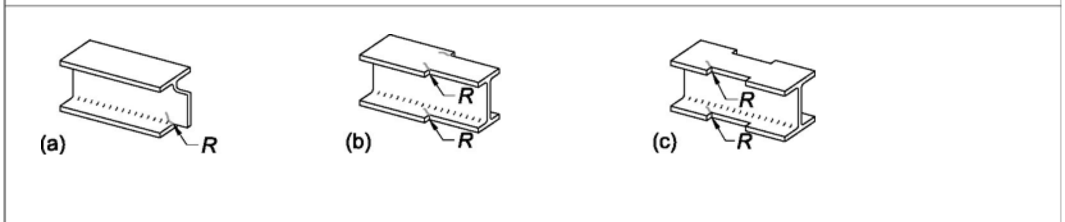
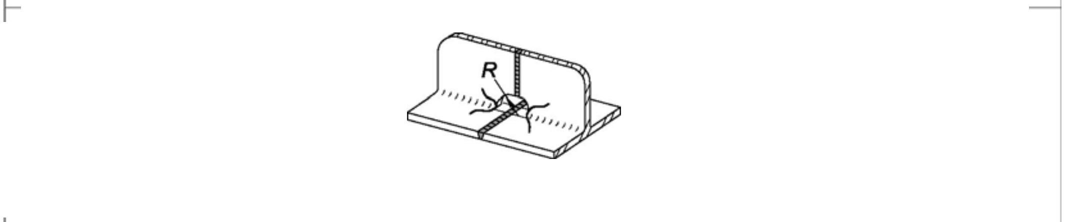
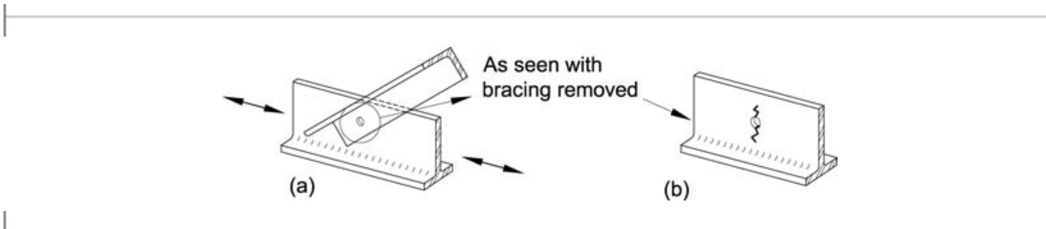
TABLE A-3.1
Fatigue Design Parameters

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 noncoated weathering steel, with as-rolled or cleaned surfaces; flame-cut edges with surface roughness value of 1,000 µin. (25 µm) or less, but without reentrant corners	A	25	24 (165)	Away from all welds or structural connections
1.2 Noncoated weathering steel base metal with as-rolled or cleaned surfaces; flame-cut edges with surface roughness value of 1,000 µin. (25 µm) or less, but without reentrant corners	B	12	16 (110)	Away from all welds or structural connections
1.3 Members with reentrant corners at copes, cuts, block-outs or other geometrical discontinuities, except weld access holes				At any external edge or at hole perimeter
$R \geq$ 1 in. (25 mm), with the radius, R, formed by predrilling, subpunching and reaming water-jet cutting or thermally cutting and grinding to a bright metal surface	C	4.4	10 (69)	
$R \geq$ 3/8 in. (10 mm) and the radius, R, formed by drilling punching, water-jet cutting, or thermal cutting; punched holes need not be reamed, and thermally cut surfaces need not be ground .	E’	0.39	2.6 (18)	

1.4 Rolled cross sections with weld access holes made to requirements of Section J1.6				At reentrant corner of weld access hole
Access hole R ≥ 1 in. (25 mm) with radius, R, formed by predrilling, subpunching and reaming or thermally cut and ground to a bright metal surface	C	4.4	10 (69)	
Access hole R ≥ 3/8 in. (10 mm) and the radius, R, need not be ground to a bright metal surface	E’	0.39	2.6 (18)	
1.5 Members with drilled or reamed holes				In net section originating at side of the hole
Holes containing pretensioned bolts	C	4.4	10 (69)	
Open holes without bolts	D	2.2	7 (48)	
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. Holes may be prepared by any method permitted by this specification.	B	12	16 (110)	Through gross section near hole
2.2 Base metal at net section of high-strength bolted joints, Holes may be prepared by any method permitted by this specificationbut thermally cut holes shall be subject to the approval of the Engineer.	B	12	16 (110)	In net section originating at side of hole
2.3 Base metal at the net section of joints with rivets, snug tightened bolts or other mechanical fasteners.	C	4.4	10 (69)	In net section originating at side of hole
2.4 Base metal at net section of eyebar head or pin plate	E	1.1	4.5 (31)	In net section originating at side of hole

TABLE A-3.1 (continued)

Fatigue Design Parameters

1.1 and 1.2	
1.3	
1.4	
1.5	

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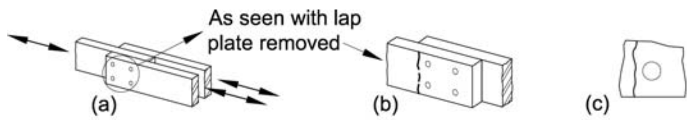
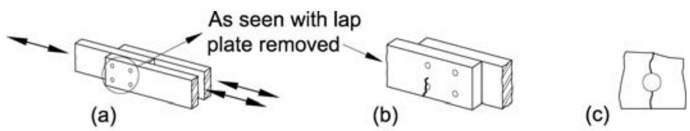


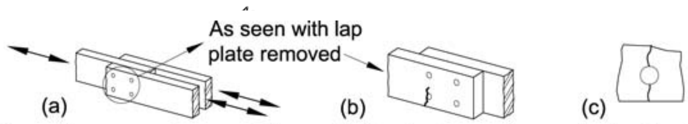
(Note: Figures are for slip-critical bolted connections.)

2.2



(Note: Figures are for bolted connections designed to bear, meeting the requirements of slip-critical connections.)

2.3



(Not~~e: Figures are for snug-tightened bolts, rivets, or other mechanical fasteners.)~~

Remove the note!

2.4

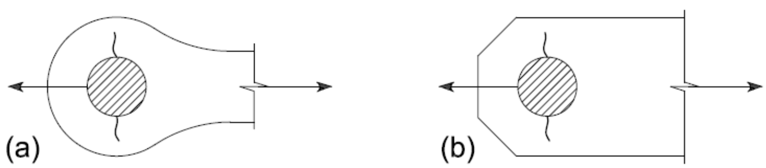


TABLE A-3.1 (continued)

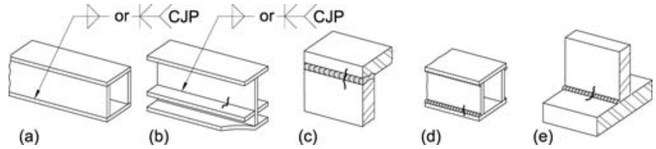
Fatigue Design Parameters

Description	Stress Catego-ry	Constant, C_f	Thresh-old, 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 CJP groove welds, back gouged and welded from second side, or by continuous fillet welds	B	12	16 (110)	From surface or internal discontinuities in weld
3.2 Base metal and weld metal in members without attachments built up of plates or shapes, connected by continuous longitudinal CJP groove welds with left-in-place continuous steel backing, or by continuous PJP groove welds	B′	6.1	12 (83)	From surface or internal discontinuities in weld
3.3 Base metal at the ends of longitudinal welds that terminate at weld access holes in connected built-up members, as well as weld toes of fillet welds that wrap around ends of weld access holes				From the weld termination into the web or flange
Access hole $R \geq$ 1 in. (25 mm) with radius, R, formed by predrilling, subpunching and reaming, or thermally cut and ground to bright metal surface	D	2.2	7 (48)	
Access hole $R \geq$ 3/8 in. (10 mm) and the radius, R, need not be ground to a bright metal surface	E’	0.39	2.6 (18)	
3.4 Base metal at ends of longitudinal intermittent fillet weld segments	E	1.1	4.5 (31)	In connected material at start and stop locations of any weld

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				In flange at toe of end weld (if present) or in flange at termination of longitudinal weld
t_f ≤ 0.8 in. (20 mm)	E	1.1	4.5 (31)	
t_f > 0.8 in. (20 mm) where t_f = thickness of member flange, in. (mm)	E′	0.39	2.6 (18)	
3.6 Base metal at ends of partial length welded coverplates or other attachments 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
t_f ≤ 0.8 in. (20 mm)	E	1.1	4.5 (31)	
t_f > 0.8 in. (20 mm)	E′	0.39	2.6 (18)	
3.7 Base metal at ends of partial length welded coverplates wider than the flange without welds across the ends				In edge of flange at end of coverplate weld
t_f ≤ 0.8 in. (20 mm)	E′	0.39	2.6 (18)	
t_f >0.8 in. (20 mm) is not permitted	None	–	–	
SECTION 4–LONGITUDINAL FILLET WELDED END CONNECTIONS				
4.1 Base metal at junction of axially loaded members with longitudinally welded end connections; welds are 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 ≤ 0.5 in. (13 mm)	E	1.1	4.5 (31)	
t > 0.5 in. (13 mm) where t = connected member thickness, as shown in Case	E′	0.39	2.6 (18)	

4.1 figure, in. (mm)				

3.1


or
CJP
or
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(a)
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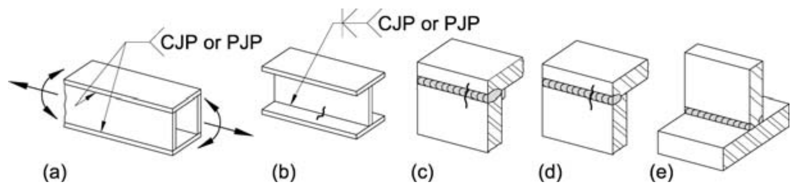
3.2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CJP or PJP
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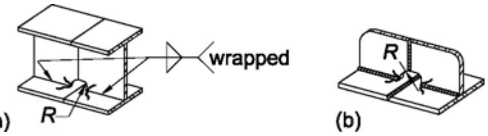
3.3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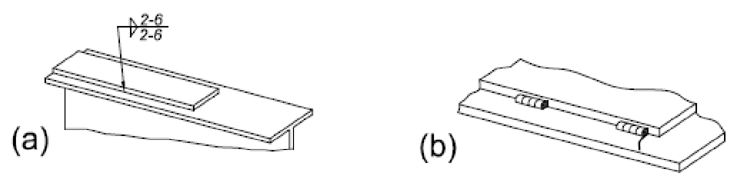
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2-6
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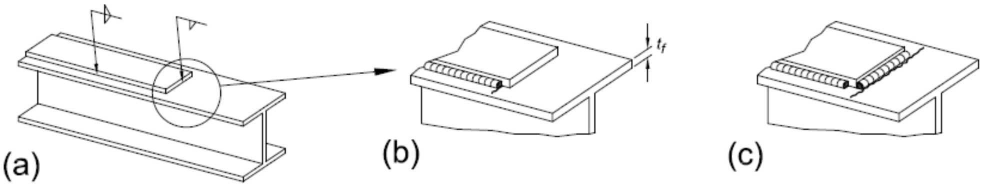
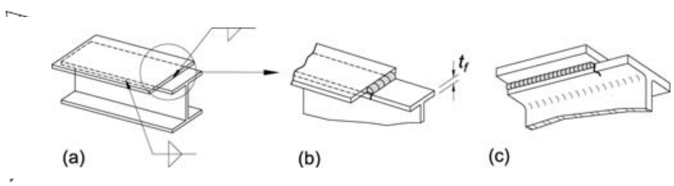
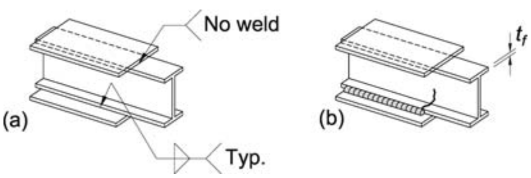
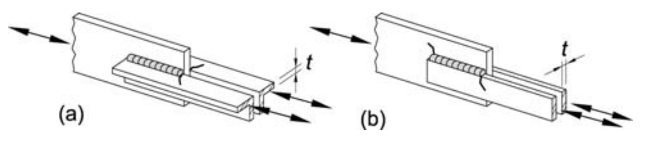
3.5	(a) (b) (c) t_f
3.6	(a) (b) (c) t_f
3.7	No weld Typ. (a) (b) t_f
4.1	(a) (b) t

TABLE A-3.1 (continued)

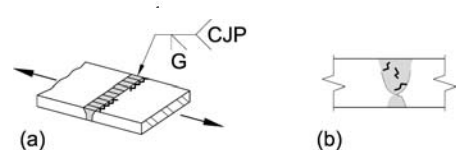
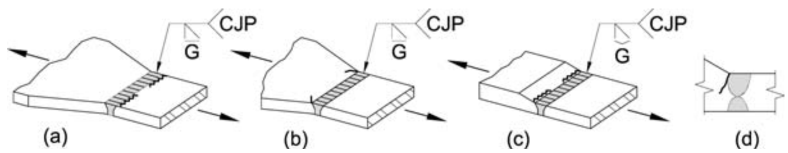
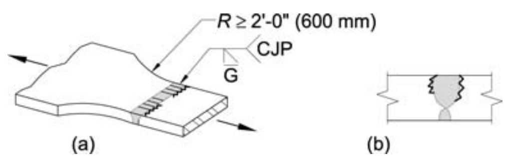
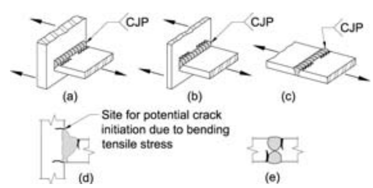
Fatigue Design Parameters

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 Weld metal and base metal in or adjacent to CJP groove welded splices in plate, rolled shapes, or built-up cross sections with no change in cross section with welds ground essentially parallel to the direction of stress and inspected in accordance with Section 3.6	B	12	16 (110)	From internal discontinuities in weld metal or along the fusion boundary
5.2 Weld metal and base metal in or adjacent to CJP 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 1:2-1/2 and inspected in accordance with Section 3.6				From internal discontinuities in metal or along the fusion boundary or at start of transition when $F_y \geq 90$ ksi (620 MPa)
$F_y < 90$ ksi (620 MPa)	B	12	16 (110)	
$F_y \geq 90$ ksi (620 MPa)	B′	6.1	12 (83)	
5.3 Base metal and weld metal in or adjacent to CJP groove welded splices with welds ground essentially parallel to the direction of stress at transitions in width made on a radius, R, of not less than 24 in. (600 mm) with the point of tangency at the end of the groove weld and inspected in accordance with Section 3.6	B	12	16 (110)	From internal discontinuities in weld metal or along the fusion boundary

5.4 Weld metal and base metal in or adjacent to CJP groove welds in T- or corner-joints or splices, without transitions in thickness or with transition in thickness having slopes no greater than 1:2-1/2, when weld reinforcement is not removed, and is inspected in accordance with Section 3.6	C	4.4	10 (69)	From weld extending into base metal or into weld metal
5.5 Base metal and weld metal in or adjacent to transverse CJP groove welded butt splices with backing left in place				From the toe of the groove weld or the toe of the weld attaching backing when applicable
Tack welds inside groove	D	2.2	7 (48)	
Tack welds outside the groove and not closer than 1/2 in. (13 mm) to the edge of base metal	E	1.1	4.5 (31)	
5.6 Base metal and weld metal at transverse end connections of tension-loaded plate elements using PJP groove welds in butt, T- or corner-joints, with reinforcing or contouring fillets; F_{SR} shall be the smaller of the toe crack or root crack allowable stress range				
Crack initiating from weld toe	C	4.4	10 (69)	Initiating from weld toe extending into base metal
Crack initiating from weld root	C′	See Eq. A-3-3 or A-3-3M	None	Initiating at weld root extending into and through weld

TABLE A-3.1 (continued)

Fatigue Design Parameters

5.1	
5.2	
5.3	
5.4	

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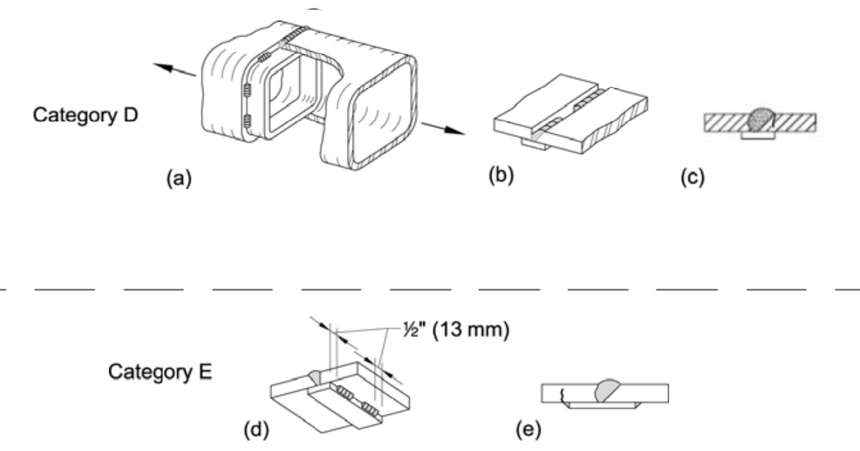

Note revised figure ©

5.6

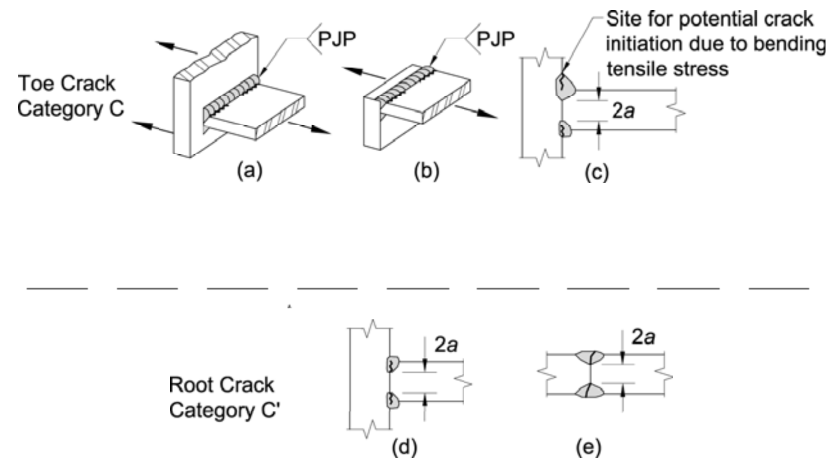

TABLE A-3.1 (continued)
Fatigue Design Parameters

Description	Stress Category	Constant C_f	Thresh-old F_{TH}, ksi (MPa)	Potential Crack Initiation Point
SECTION 5–WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS (cont'd)				
5.7 Base metal and weld 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 weld toe crack or weld root crack allowable stress range				
Crack initiating from weld toe	C	4.4	10 (69)	Initiating from weld toe extending into base metal
Crack initiating from weld root	C″	See Eq. A-3-5 or A-3-5M	None	Initiating at weld root extending into and through weld
5.8 Base metal of tension-loaded plate elements, and on built-up shapes and rolled beam webs or flanges at toe of transverse fillet welds adjacent to welded transverse stiffeners	C	4.4	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 of equal or unequal thickness at details attached by CJP groove welds subject to longitudinal loading only when the detail embodies a transition radius, *R*, with the weld termination ground smooth and inspected in accordance with Section 3.6				Near point of tangency of radius at edge of member
$R \geq$ 24 in. (600 mm)	B	12	16 (110)	
6 in. ≤ *R* < 24 in. (150 mm ≤ *R* < 600 mm)	C	4.4	10 (69)	
2 in. ≤ *R* < 6 in. (50 mm ≤ *R* < 150 mm)	D	2.2	7 (48)	
R < 2 in. (50 mm)	E	1.1	4.5 (31)	

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TABLE A-3.1 (continued)
Fatigue Design Parameters

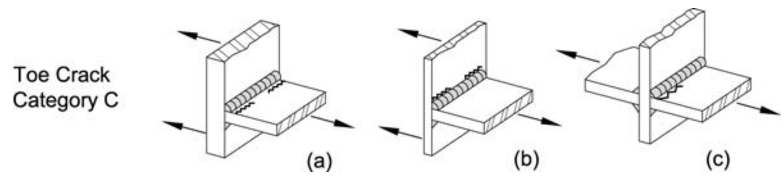
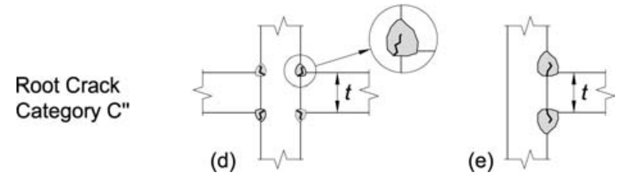
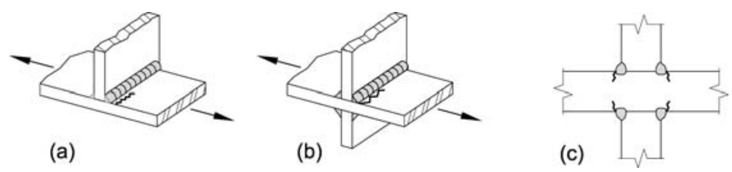
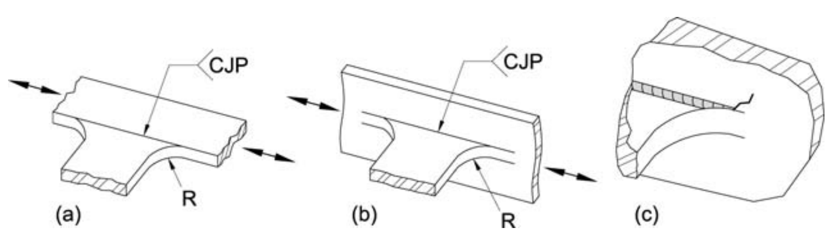
5.7	
	
5.8	
6.1	

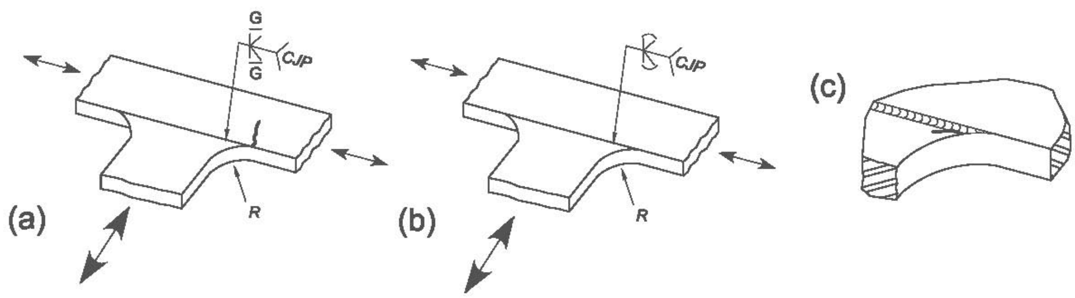
TABLE A-3.1 (continued)
Fatigue Design Parameters

Description	Stress Category	Constant C_f	Threshold F_{TH}, ksi (MPa)	Potential Crack Initiation Point
SECTION 6–BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (continued)				
6.2 Base metal at details of equal thickness attached by CJP 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 and inspected in accordance with Section 3.6				
(a) When weld reinforcement is removed				Near point of tangency of radius or in the weld or at fusion boundary or member or attachment
$R \geq$ 24 in. (600 mm)	B	12	16 (110)	
6 in. $\leq R <$ 24 in. (150 mm $\leq R <$ 600 mm)	C	4.4	10 (69)	
2 in. $\leq R <$ 6 in. (50 mm $\leq R <$ 150 mm)	D	2.2	7 (48)	
$R <$ 2 in. (50 mm)	E	1.1	4.5 (31)	
(b) When weld reinforcement is not removed				At toe of the weld either along edge of member or the attachment
$R \geq$ 6 in. (150 mm)	C	4.4	10 (69)	
2 in. $\leq R <$ 6 in. (50 mm $\leq R <$ 150 mm)	D	2.2	7 (48)	
$R <$ 2 in. (50 mm)	E	1.1	4.5 (31)	

6.3 Base metal at details of unequal thickness attached by CJP 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 and in accordance with Section 3.6:				
(a) When weld reinforcement is removed				
R > 2 in. (50 mm)	D	2.2	7 (48)	At toe of weld along edge of thinner material
R ≤ 2 in. (50 mm)	E	1.1	4.5 (31)	In weld termination in small radius
(b) When reinforcement is not removed				
Any radius	E	1.1	4.5 (31)	At toe of weld along edge of thinner material

TABLE A-3.1 (continued)
Fatigue Design Parameters

6.2



6.3

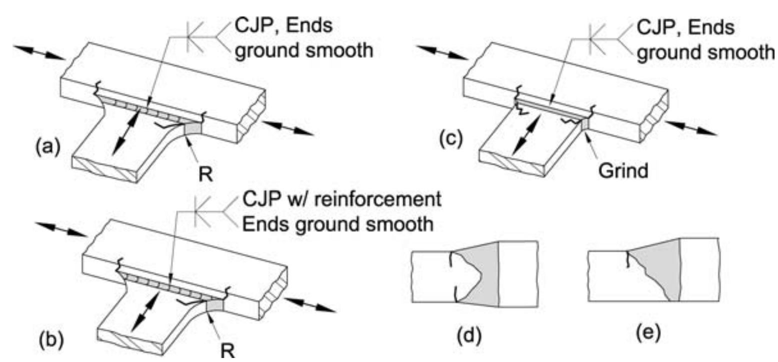


TABLE A-3.1 (continued)
Fatigue Design Parameters

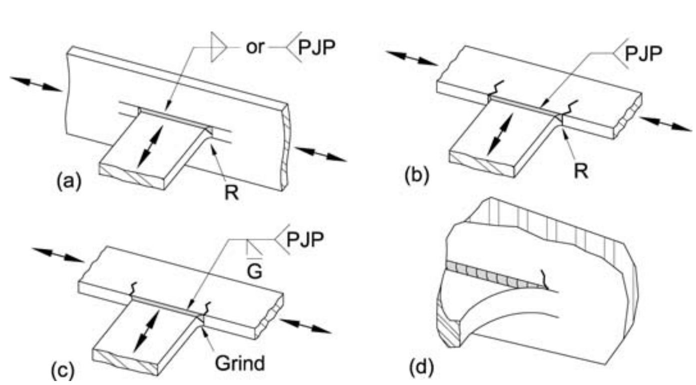
Description	Stress Category	Constant C_f	Threshold F_{TH}, ksi (MPa)	Potential Crack Initiation Point
SECTION 6–BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (continued)				
6.4 Base metal of equal or unequal thickness, subject to longitudinal stress at transverse members, with or without transverse stress, attached by fillet or PJP groove welds parallel to direction of stress when the detail embodies a transition radius, *R*, with weld termination ground smooth				Initiating in base metal at the weld termination or at the toe of the weld extending into the base metal
R > 2 in. (50 mm)	D	2.2	7 (48)	
$R \le$ 2 in. (50 mm)	E	1.1	4.5 (31)	
SECTION 7–BASE METAL AT SHORT ATTACHMENTS[a]				

7.1 Base metal subject to longitudinal loading at details with welds parallel or transverse to the direction of stress, with or without transverse load on the detail, where the detail embodies no transition radius, *R*, and with detail length, *a*, in direction of stress and thickness of the attachment, *b*:				Initiating in base metal at the weld termination or at the toe of the weld extending into the base metal
a < 2 in. (50 mm) for any thickness, b	C	4.4	10 (69)	
2 in. (50 mm) ≤ a ≤ 4 in. (100 mm) and a≤ 12b	D	2.2	7 (48)	
2 in. (50 mm) ≤ a ≤ 4 in. (100 mm) and a > 12b *a> 4 in. (100 mm) and b≤ 0.8 in. (20 mm)*	E	1.1	4.5 (31)	
a > 4 in. (100 mm) and b > 0.8 in. (20 mm)	E′	0.39	2.6 (18)	
7.2 Base metal subject to longitudinal stress at details attached by fillet or PJP groove welds, with or without transverse load on detail, when the detail embodies a transition radius, *R*, with weld termination ground smooth:				Initiating in base metal at the weld termination, extending into the base metal
R > 2 in. (50 mm)	D	2.2	7 (48)	
R ≤ 2 in. (50 mm)	E	1.1	4.5 (31)	
[a] "Attachment," as used herein, is defined as any steel detail welded to a member that causes a deviation in the stress flow in the member and, thus, reduces the fatigue resistance. The reduction is due to the presence of the attachment, not due to the loading on the attachment.				

Align the descriptions, Ctegories, Cfs and Fths after the chnges are accepted

TABLE A-3.1 (continued)
Fatigue Design Parameters

6.4



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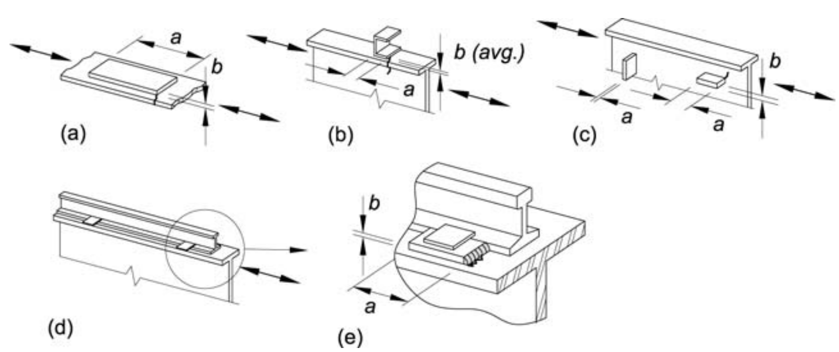

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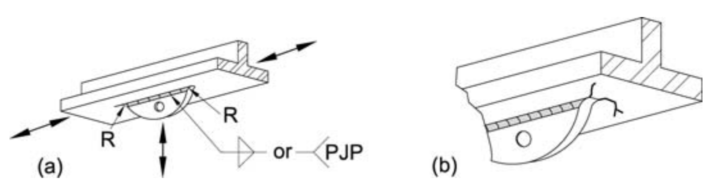


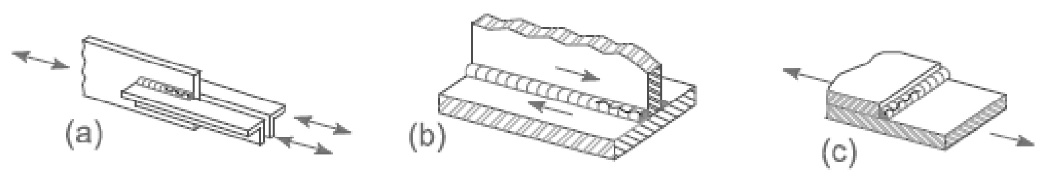
TABLE A-3.1 (continued)
Fatigue Design Parameters

Description	Stress Category	Constant C_f	Threshold F_{TH}, ksi (MPa)	Potential Crack Initiation Point
SECTION 8–MISCELLANEOUS				
8.1 Base metal at steel headed stud anchors attached by fillet weld or automatic stud welding	C	4.4	10 (69)	At toe of weld in base metal
8.2 Shear on throat of any fillet weld, continuous or intermittent, longitudinal or transverse	F	See Eq. A-3-2 or A-3-2M	See Eq. A-3-2 or A-3-2M	Initiating at the root of the fillet weld, extending into the weld
8.3 Base metal at plug or slot welds	E	1.1	4.5 (31)	Initiating in the base metal at the end of the plug or slot weld, extending into the base metal
8.4 Shear on plug or slot welds	F	See Eq. A-3-2 or A-3-2M	See Eq. A-3-2 or A-3-2M	Initiating in the weld at the faying surface, extending into the weld
8.5 High-strength bolts, common bolts, threaded anchor rods, and hanger rods, whether pretensioned in accordance with Table J3.1 or J3.1M, or snug-tightened with cut, ground or rolled threads; stress range on tensile stress area due to applied cyclic load plus prying action, when applicable	G	0.39	7 (48)	Initiating at the root of the threads, extending into the fastener

TABLE A-3.1 (continued)
Fatigue Design Parameters

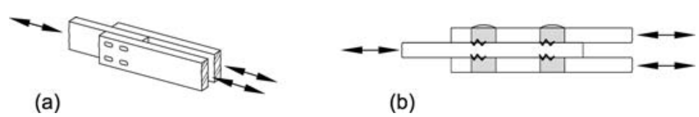
8.1

8.2



8.3

8.4



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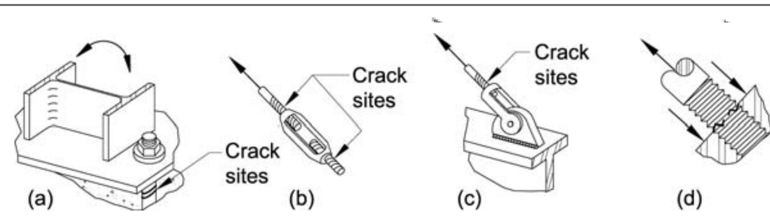

APPENDIX 4

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.

> **User Note:** Throughout this chapter, the term "elevated temperatures" refers to temperatures due to unintended fire exposure only.

The appendix is organized as follows:

4.1. General Provisions
4.2. Structural Design for Fire Conditions by Analysis
4.3. Design by Qualification Testing

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.

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 evaluation 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.

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 applicable building code (ABC).

Structural design for fire conditions using Appendix 4.2 shall be performed using the load and resistance factor design method in accordance with the provisions of Section B3.1 (LRFD).

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 the ABC.

4. Load Combinations and Required Strength

In the absence of ABC provisions for design under fire exposures, the required strength of the structure and its elements shall be determined from the gravity load combination as follows:

$$(0.9 \text{ or } 1.2)\, D + A_T + 0.5L + 0.2S \tag{A-4-1}$$

where

A_T = nominal forces and deformations due to the design-basis fire defined in Section 4.2.1

D = nominal dead load

L = nominal occupancy live load

S = nominal snow load

User Note: ASCE/SEI 7 Section 2.5 contains this load combination for extraordinary events, which includes fire. Live load reduction is permitted.

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.

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.

The analysis methods in Section 4.2 shall be used in accordance with the provisions for alternative materials, designs, and methods as permitted by the ABC. When the analysis methods in Section 4.2 are used to demonstrate equivalency to hourly ratings based on qualification testing in Section 4.3, the design-basis fire shall be permitted to be determined in accordance with ASTM E119.

1a. 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.

1b. 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 of the space (natural and mechanical), compartment dimensions, and thermal characteristics of the compartment boundary.

The fire duration in a particular area shall be determined from the total combustible mass, or fuel load in the space. In the case of either a localized fire or a post-flashover compartment fire, the fire duration shall be determined as the total combustible mass divided by the mass loss rate.

1c. Exterior Fires

The exposure effects of the exterior structure to flames projecting from windows or other wall openings as a result of a post-flashover compartment fire shall be addressed 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 4.2.1b shall be used for describing the characteristics of the interior compartment fire.

1d. Active Fire-Protection Systems

The effects of active fire-protection systems shall be addressed 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.

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.

3. Material Properties at Elevated Temperatures

The effects of elevated temperatures on the physical and mechanical properties of materials shall be considered in the analysis and design of structural members, components and systems. Any rational method that establishes material properties at elevated temperatures that is based on test data is permitted, including the methods defined in Sections 4.2.3a, and 4.2.3b.

3a. Thermal Elongation

The coefficients of thermal expansion shall be taken as follows:

(a) For structural and reinforcing steels: For calculations at temperatures above 150°F (66°C), the coefficient of thermal expansion is 7.8×10^{-6}/°F (1.4×10^{-5}/°C).

(b) For normal weight concrete: For calculations at temperatures above 150°F (66°C), the coefficient of thermal expansion is 10×10^{-6}/°F (1.8×10^{-5}/°C).

(c) For lightweight concrete: For calculations at temperatures above 150°F (66°C), the coefficient of thermal expansion is 4.4×10^{-6}/°F (7.9×10^{-6}/°C).

3b. Mechanical Properties of Structural Steel, Hot-Rolled Reinforcing Steel, and Concrete at Elevated Temperatures

The uniaxial engineering stress-strain-temperature relationship for structural steel, hot rolled reinforcing steel, and concrete shall be determined using this section. This applies only to structural and reinforcing steels with a specified minimum yield strength, F_y, equal to 65 ksi (450 MPa) or less, and to concrete with a specified compressive strength, f'_c, equal to 8 ksi (55 MPa) or less.

(a) Structural and Hot Rolled Reinforcing Steel

Table A-4.2.1 provides retention factors (k_E, k_y, and k_p) for steel which are expressed as the ratio of the mechanical property at elevated temperature with respect to the property at ambient, assumed to be 68°F (20°C). It is permitted to interpolate between these values. The properties at elevated temperature, T, and are defined as follows:

$E(T)$ is the modulus of elasticity of steel at elevated temperature, ksi (MPa), which is calculated as a ratio to the ambient property as specified in Table A-4.2.1.

$G(T)$ is the shear modulus of elasticity of steel at elevated temperature, ksi (MPa) , which is calculated as a ratio to the ambient property as specified in Table A-4.2.1.

$F_y(T)$ is the specified minimum yield stress of steel at elevated temperature, ksi (MPa) , which is calculated as a ratio to the ambient property as specified in Table A-4.2.1.

$F_p(T)$ is the proportional limit at elevated temperature, which is calculated as a ratio to yield strength as specified in Table A-4.2.1.

$F_u(T)$ is the specified minimum tensile strength at elevated temperature, which is equal to $F_y(T)$ for temperatures greater than 750°F (400°C). For temperatures less than or equal to 750°F (400°C), F_u may be used in place of $F_u(T)$.

The engineering stress at elevated temperature, $F(T)$, at each strain range shall be determined as follows:

(a) When in the elastic range [$\varepsilon(T) < \varepsilon_p(T)$]

$$F(T) = E(T)\,\varepsilon(T) \qquad \text{(A-4-2)}^{[a]}$$

(b) When in the nonlinear range [$\varepsilon_p(T) \le \varepsilon(T) \le \varepsilon_y(T)$]

$$F(T) = F_p(T) - c + \frac{b}{a}\sqrt{a^2 - \left[\varepsilon_y(T) - \varepsilon(T)\right]^2} \qquad \text{(A-4-3)}^{[a]}$$

(c) When in the plastic range [$\varepsilon_y(T) \le \varepsilon(T) \le \varepsilon_u(T)$]

$$F(T) = F_y(T) \qquad \text{(A-4-4)}^{[a]}$$

where

$\varepsilon(T)$ = the engineering strain at elevated temperature, in./in. (m/m)

$\varepsilon_p(T)$ = the engineering strain at the proportional limit at elevated temperature, in./in. (m/m) = $F_p(T) / E(T)$

$\varepsilon_y(T)$ = the engineering yield strain at elevated temperature, in./in. (m/m) = 0.02 in./in. (m/m)

$$a^2 = \left[\varepsilon_y(T) - \varepsilon_p(T)\right]\left[\varepsilon_y(T) - \varepsilon_p(T) + \frac{c}{E(T)}\right] \qquad \text{(A-4-5)}^{[a]}$$

$$b^2 = E(T)\left[\varepsilon_y(T) - \varepsilon_p(T)\right]c + c^2 \qquad \text{(A-4-6)}^{[a]}$$

$$c = \frac{\left[F_y(T) - F_p(T)\right]^2}{E(T)\left[\varepsilon_y(T) - \varepsilon_p(T)\right] - 2\left[F_y(T) - F_p(T)\right]} \qquad \text{(A-4-7)}^{[a]}$$

User Note: The equation for the plastic range conservatively neglects the strain-hardening portion, but strain-hardening is permitted to be includedThe plateau of the plastic range does not exceed the ultimate strain, $\varepsilon_u(T)$, where $\varepsilon_u(T)$ = 15%.

User Note: This section applies to structural steel materials in Section A3.1 and to hot-rolled reinforcing steel with a specified minimum yield strength, F_y, equal to 65 ksi or less, which includes ASTM A615/A615M Gr. 60 (420) and ASTM A706/A706M Gr. 60 (420) steel reinforcement.

(b) Concrete

Table A-4.2.2 provides retention factors for concrete which are expressed as the ratio of the mechanical property at elevated temperature with respect to the property at ambient, assumed to be 68°F (20 °C). It is permitted to interpolate between these values. For lightweight concrete, values of $\varepsilon_{cu}(T)$ shall be obtained from tests. The properties at elevated temperature, T, are defined as follows:

[a] EC4, European Committee for Standardization (CEN), Eurocode 4 Design of Composite Steel and Concrete Structures: Part 1.2: General Rules, Structural Fire Design, EN 1994-1-2, CEN, Brussels, 2005.

$f_c'(T)$ = the specified compressive strength of concrete at elevated temperature, ksi (MPa), which is calculated as a ratio to the ambient property as specified in Table A-4.2.2.

$E_c(T)$ = the modulus of elasticity of concrete at elevated temperature, ksi (MPa)

$\varepsilon_{cu}(T)$ = the concrete strain corresponding to $f_c'(T)$ at elevated temperature, in./in. (m/m)

The uniaxial stress-strain-temperature relationship for concrete in compression is permitted to be calculated as follows:

$$F_c(T) = f_c'(T)\left\{\frac{3\left[\dfrac{\varepsilon_c(T)}{\varepsilon_{cu}(T)}\right]}{2+\left[\dfrac{\varepsilon_c(T)}{\varepsilon_{cu}(T)}\right]^3}\right\} \qquad \text{(A-4-8)[a]}$$

where $F_c(T)$ and $\varepsilon_c(T)$ are the concrete compressive stress and strain, respectively, at elevated temperature.

User Note: The tensile strength of concrete at elevated temperature can be taken as zero, or not more than 10% of the compressive strength at the corresponding temperature.

(c) Strengths of Bolts at Elevated Temperatures

Table A-4.2.3 provides retention factors for high-strength bolts which are expressed as the ratio of the mechanical property at elevated temperature with respect to the property at ambient, which is assumed to be 68°F (20°C). The properties at elevated temperature, T, are defined as follows:

$F_{nt}(T)$ = nominal tensile strength of the bolt, ksi (MPa)
$F_{nv}(T)$ = nominal shear strength of the bolt, ksi (MPa)

TABLE A-4.2.1
Properties of Steel at Elevated Temperatures

Steel Temperature, °F (°C)	$k_E = E(T)/E = G(T)/G$	$k_p = F_p(T)/F_y$	$k_y = F_y(T)/F_y$
68 (20)	1.00	1.00	1.00
200 (93)	1.00	1.00	1.00
400 (200)	0.90	0.80	1.00
600 (320)	0.78	0.58	1.00
750 (400)	0.70	0.42	1.00
800 (430)	0.67	0.40	0.94
1000 (540)	0.49	0.29	0.66
1200 (650)	0.22	0.13	0.35

[a] EC4, European Committee for Standardization (CEN), Eurocode 4 Design of Composite Steel and Concrete Structures: Part 1.2: General Rules, Structural Fire Design, EN 1994-1-2, CEN, Brussels, 2005.

1400 (760)	0.11	0.06	0.16
1600 (870)	0.07	0.04	0.07
1800 (980)	0.05	0.03	0.04
2000 (1100)	0.02	0.01	0.02
2200 (1200)	0.00	0.00	0.00

TABLE A-4.2.2
Properties of Concrete at Elevated Temperatures

Concrete Temperature, °F (°C)	$k_c = f'_c(T)/f'_c$		$E_c(T)/E_c$	$\varepsilon_{cu}(T)$, %
	Normal Weight Concrete	Lightweight Concrete		Normal Weight Concrete
68 (20)	1.00	1.00	1.00	0.25
200 (93)	0.95	1.00	0.93	0.34
400 (200)	0.90	1.00	0.75	0.46
550 (290)	0.86	1.00	0.61	0.58
600 (320)	0.83	0.98	0.57	0.62
800 (430)	0.71	0.85	0.38	0.80
1000 (540)	0.54	0.71	0.20	1.06
1200 (650)	0.38	0.58	0.092	1.32
1400 (760)	0.21	0.45	0.073	1.43
1600 (870)	0.10	0.31	0.055	1.49
1800 (980)	0.05	0.18	0.036	1.50
2000 (1100)	0.01	0.05	0.018	1.50
2200 (1200)	0.00	0.00	0.000	0.00

TABLE A-4.2.3
Properties of Group 120 and Group 150 High-Strength Bolts at Elevated Temperatures

Bolt Temperature, °F (°C)	$F_{nt}(T)/F_{nt}$ or $F_{nv}(T)/F_{nv}$
68 (20)	1.00
200 (93)	0.97
300 (150)	0.95
400 (200)	0.93
600 (320)	0.88
800 (430)	0.71
900 (480)	0.59
1000 (540)	0.42
1200 (650)	0.16
1400 (760)	0.08
1600 (870)	0.04
1800 (980)	0.01
2000 (1100)	0.00

4. Structural Design Requirements

4a. General Requirements

The structural frame and foundation shall be capable of providing the 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. Frame stability and required strength shall be determined in accordance with the requirements of Section C1.

Continuous load paths shall be provided to transfer all forces from the exposed region to the final point of resistance.

The requirement for steam vent holes in concrete-filled composite members shall be evaluated. Any rational method that considers heat transfer through the cross-section, water content in concrete, fire protection, and the allowable pressure build up in the member is permitted for calculating the size and spacing of vent holes.

User Note: Section 4.3.2.2.1 provides a possible vent hole configuration for concrete-filled columns.

4b. 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 have the design strength necessary to resist the shears, axial forces and moments determined in accordance with these provisions.

Connections shall be designed and detailed to resist the imposed loading and deformation demands during a design-basis fire as required to meet the performance objectives stated in Section 4.1.1. Where the means of providing fire resistance requires the evaluation of deformation criteria, the deformation of the structural system, or members thereof, under the design-basis fire shall not exceed the prescribed limits.

User Note: Typical simple shear connections may need additional design enhancements for ductility and resistance to large compression and tensile forces that may develop during the design-basis fire exposure. A fire exposure will not only affect the magnitude of member end reactions, but may also change the nature of the reaction to a limit state different from the controlling mode at ambient temperature.

It shall be permitted to include membrane action of composite floor slabs for fire resistance if the design provides for the effects of increased connection tensile forces and redistributed gravity load demands on the adjacent framing supports.

4c. Design by Advanced Methods of Analysis

Design by advanced methods of analysis is 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 deformations in the structural system due 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, inelastic behavior and load redistribution, large deformations, time-dependent effects such as creep, and uncertainties resulting from variability in material properties at elevated temperature. Support and restraint conditions (forces, moments, and boundary conditions) shall represent the behavior of the structure during a design-basis fire. Material properties shall be defined as per Section 4.2.3.

The resulting analysis shall address all relevant limit states, such as excessive deflections, connection ruptures, and global or local buckling.

4d. Design by Simple Methods of Analysis

The methods of analysis in this section are permitted to be used for the evaluation of the performance of individual members at elevated temperatures during exposure to a design-basis fire. When evaluating individual members, the support and restraint conditions (forces, moments and boundary conditions) applicable at normal temperatures are permitted to be assumed to remain unchanged throughout the fire exposure.

For evaluating the performance of structural frames during exposure to a design-basis fire, member demands (forces and moments) are also permitted to be determined through consideration of reduced stiffness at elevated temperatures, appropriate boundary conditions, and thermal deformations.

It is permitted to model the thermal response of steel and composite members using a lumped heat capacity analysis with heat input as determined by the design-basis fire defined in Section 4.2.1, using the temperature equal to the maximum steel temperature. For composite beams, the maximum steel temperature shall be assigned to the bottom flange and a temperature gradient shall be applied to incorporate thermally induced moments, as stipulated in Section 4.2.4d(f) .

For steel temperatures less than or equal to 400°F (200°C), the member and connection design strengths is permitted to be determined without consideration of temperature effects on the nominal strengths.

The design strength shall be determined as in Section B3.1. The nominal strength, R_n, shall be calculated using material properties, as provided in Section 4.2.3b, at the temperature developed by the design-basis fire and as stipulated in Sections 4.2.4d(a) through (f).

User Note: Lumped heat capacity analysis assumes uniform temperature over the section and length of the member, which is generally a reasonable assumption for many structural members exposed to post-flashover fires. Consideration should be given to the use of the uniform temperature assumption as it may not always be applicable or conservative.

At temperatures below 400°F (200°C), the reduction in steel properties need not be considered in calculating member strengths despite small reductions in material properties in Table A-4.2.1. This is a simplifying assumption used only in the simple method of analysis.

(a) Design for Tension

The nominal strength for tension shall be determined using the provisions of Chapter D, with steel properties as stipulated in Section 4.2.3b(a) and assuming a uniform temperature over the cross section using the temperature equal to the maximum steel temperature.

(b) Design for Compression

For nonslender-element columns, the nominal strength for flexural buckling of compression members shall be determined using the provisions of Chapter E with steel properties as stipulated in Section 4.2.3b(a).Equation A-4-9 shall be used in lieu of Equations E3-2 and E3-3 to calculate the nominal compressive strength for flexural buckling:

$$F_{cr}(T) = \left[0.42^{\sqrt{\frac{F_y(T)}{F_e(T)}}} \right] F_y(T) \qquad \text{(A-4-9)}$$

where $F_y(T)$ is the yield stress at elevated temperature and $F_e(T)$ is the critical elastic buckling stress calculated from Equation E3-4 with the elastic modulus, $E(T)$, at elevated temperature. $F_y(T)$ and $E(T)$ are obtained using coefficients from Table A-4.2.1.

The strength of leaning (gravity) columns may be increased by rotational restraints from cooler columns in the stories above and below the story exposed to the fire. This increased strength applies to fires on only one floor and should not be used for multiple story fires. The increase in design strength can be accounted for by reducing the column slenderness (L_c/r) used to calculate $F_e(T)$ in Equation A-4-9 to $(L_c/r)_T$ as follows:

$$\left(\frac{L_c}{r}\right)_T = \left(1 - \frac{T-32}{n(3{,}600)}\right)\left(\frac{L_c}{r}\right) - \frac{35}{n(3{,}600)}(T-32) \geq 0 \quad (^{\circ}\mathrm{F}) \qquad \text{(A-4-10)}$$

$$\left(\frac{L_c}{r}\right)_T = \left(1 - \frac{T}{n(2{,}000)}\right)\left(\frac{L_c}{r}\right) - \frac{35T}{n(2{,}000)} \geq 0 \quad (^{\circ}\mathrm{C}) \qquad \text{(A-4-10M)}$$

where
- L_c = KL = effective length of member, in. (mm)
- L = laterally unbraced length of the member, in. (mm)
- K = effective length factor
- r = radius of gyration, in. (mm)
- T = steel temperature, °F , °C
- n = 1 for columns with cooler columns both above and below
- n = 2 for columns with cooler columns either above or below only

User Note: The design equations for compression predict flexural buckling capacities of wide flange rolled shapes, but do not consider local buckling and torsional buckling. If applicable, these additional limit states must be considered with an alternative method. For most fire conditions, uniform heating and temperatures govern the design for compression. When uniform heating is not a reasonable assumption, alternative methods must be used to account for the effects of nonuniform heating and resulting thermal gradients on the design strength of compression members, as the simple method assumes a uniform temperature distribution.

(c) Design for Compression in Concrete-Filled Composite Columns

For concrete-filled composite columns, the nominal strength for compression shall be determined using the provisions of Section I2.2 with steel and concrete properties as stipulated in Section A-4.2.3b. Equation A-4-11 shall be used in lieu of Equations I2-2 and I2-3 to calculate the nominal compressive strength for flexural buckling:

$$P_n(T) = \left[0.45^{\left(\frac{P_{no}(T)}{P_e(T)}\right)^{0.3}}\right] P_{no}(T) \qquad \text{(A-4-11)}$$

where $P_{no}(T)$ is calculated at elevated temperature using Equations I2-

10, I2-11, and I2-12. $P_e(T)$ is calculated at elevated temperature using Equations I2-5, I2-13, and I2-14. . $F_y(T)$, $f'_c(T)$, $E_s(T)$, and $E_c(T)$ are obtained using coefficients from Tables A-4.2.1 and A-4.2.2.

(d) Design for Compression in Concrete-Filled Composite Plate Shear Walls

For concrete-filled composite plate shear walls, the nominal strength for compression shall be determined using the provisions of Section I2.3 with steel and concrete properties as stipulated in Section A-4.2.3b and Equation A-4-12 used in lieu of Equations I2-2 and I2-3 to calculate the nominal compressive strength for flexural buckling:

$$P_n(T) = \left[0.32^{\left(\frac{P_{no}(T)}{P_e(T)}\right)^{0.3}}\right] P_{no}(T) \qquad \text{(A-4-12)}$$

where $P_{no}(T)$ is calculated at elevated temperature using Equation I2-16. $P_e(T)$ is calculated at elevated temperature using Equations I2-5 and I1-1. $F_y(T)$, $f'_c(T_c)$, $E_s(T)$, and $E_c(T_c)$ are obtained using coefficients from Tables A-4.2.1 and A-4.2.2.

User Note: For composite members, the steel temperature is determined using heat transfer equations with heat input corresponding to the design-basis fire. The temperature distribution in concrete infill can be calculated using one- or two-dimensional heat transfer equations. The regions of concrete infill will have varying temperatures and mechanical properties. Concrete contribution to axial strength and effective stiffness can therefore be calculated by discretizing the cross-section into smaller elements (with each concrete element considered to have a uniform temperature) and summing up the contribution of individual elements.

(e) Design for Flexure

For steel beams, the calculated bottom flange temperature shall be constant over the depth of the member.

(1) The nominal strength for flexure shall be determined using the provisions of Chapter F with steel properties as stipulated in Section 4.2.3b(b).Equations A-4-13 through A-4-19 shall be used in lieu of Equations F2-2 through F2-6 to calculate the nominal flexural strength for lateral-torsional buckling of doubly symmetric compact rolled wide-flange shapes bent about their major axis: When $L_b \le L_r(T)$

$$M_n(T) = C_b\left\{F_L(T)S_x + \left[M_p(T) - F_L(T)S_x\right]\left[1 - \frac{L_b}{L_r(T)}\right]^{c_x}\right\} \le M_p(T) \qquad \text{(A-4-13)}$$

(2) When $L_b > L_r(T)$

$$M_n(T) = F_{cr}(T)S_x \le M_p(T) \qquad \text{(A-4-14)}$$

where

$$F_{cr}(T) = \frac{C_b \pi^2 E(T)}{\left(\frac{L_b}{r_{ts}}\right)^2} \sqrt{1 + 0.078 \frac{Jc}{S_x h_o} \left(\frac{L_b}{r_{ts}}\right)^2} \qquad \text{(A-4-15)}$$

$$L_r(T) = 1.95 r_{ts} \frac{E(T)}{F_L(T)} \sqrt{\frac{Jc}{S_x h_o} + \sqrt{\left(\frac{Jc}{S_x h_o}\right)^2 + 6.76 \left[\frac{F_L(T)}{E(T)}\right]^2}}$$

(A-4-16)

$$F_L(T) = F_y\left(k_p - 0.3k_y\right) \qquad \text{(A-4-17)}$$

$$M_p(T) = F_y(T) Z_x \qquad \text{(A-4-18)}$$

$$c_x = 0.53 + \frac{T}{450} \leq 3.0 \text{ where } T \text{ is in °F} \qquad \text{(A-4-19)}$$

$$c_x = 0.6 + \frac{T}{250} \leq 3.0 \text{ where } T \text{ is in °C} \qquad \text{(A-4-19M)}$$

and

T = elevated temperature of steel due to unintended fire, °F (°C)

The material properties at elevated temperatures, $E(T)$ and $F_y(T)$, and the k_p and k_y coefficients are calculated in accordance with Table A-4.2.1, and other terms are as defined in Chapter F.

User Note: $F_L(T)$ represents the initial yield stress, which assumes a residual stress of $0.3F_y$. Alternatively, 10 ksi (69 MPa) may be used in place of $0.3F_y$ for calculation of $F_L(T)$.

User Note: The equations for lateral-torsional buckling do not consider local buckling. If applicable, the effects of local buckling must be considered with an alternative method.

(f) Design for Flexure in Composite Beams

For composite beams, the calculated bottom flange 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% from the mid-depth of the web to the top flange of the beam.

The nominal strength of a composite flexural member shall be determined using the provisions of Chapter I, with reduced yield stresses in the steel as determined from Table A-4.2.1. Steel properties will vary as the temperature along the depth of section changes.

Alternatively, the nominal flexural strength of a composite beam, $M_n(T)$, is permitted to be calculated using the bottom flange temperature, T, as follows:

$$M_n(T) = r(T) M_n \qquad \text{(A-4-20)}$$

where,

M_n = nominal flexural strength at ambient temperature calculated in accordance with provisions of Chapter I, kip-in. (N-mm)

$r(T)$ = retention factor depending on bottom flange temperature, T, as given in Table A-4.2.4

TABLE A-4.2.4
Retention Factor for Composite Flexural Members

Bottom Flange Temperature, °F (°C)	$r(T)$
68 (20)	1.00
300 (150)	0.98
600 (320)	0.95
800 (430)	0.89
1000 (540)	0.71
1200 (650)	0.49
1400 (760)	0.26
1600 (870)	0.12
1800 (980)	0.05
2000 (1100)	0.00

(g) Design for Shear

The nominal strength for shear yielding shall be determined in accordance with the provisions of Chapter G, with steel properties as stipulated in Section 4.2.3b(a) and assuming a uniform temperature over the cross section.

User Note: Shear yielding equations do not consider shear buckling or tension field action. If applicable, these limit states must be considered with an alternative method.

(h) Design for Combined Forces and Torsion

The nominal strength for combinations of axial force and flexure about one or both axes, with or without torsion, shall be in accordance with the provisions of Chapter H with the design axial and flexural strengths as stipulated in Sections 4.2.4d(a) to (d). Nominal strength for torsion shall be determined in accordance with the provisions of Chapter H, with the steel properties as stipulated in Section 4.2.3b(a), assuming uniform temperature over the cross section.

4e. Design by Critical Temperature Method

The critical temperature of a structural member is the temperature at which the demand on the member exceeds its capacity under fire conditions. The evaluation methods in this section are permitted to be used in lieu of Section 4.2.4d for tension members, continuously braced beams not supporting concrete slabs, or compression members that are assumed to be simply supported and develop a uniform temperature over the cross section throughout the fire exposure.

The use of the critical temperature methods shall be limited to steel members with wide-flange rolled shapes which have nonslender elements per Section B4.

(a) Design for Tensile Yielding

The critical temperature of a tension member is permitted to be calculated as follows:

$$T_{cr} = 816 - 306 \ln\left(\frac{R_u}{R_n}\right) \text{ in °F} \quad \text{(A-4-21)}$$

$$T_{cr} = 435 - 170 \ln\left(\frac{R_u}{R_n}\right) \text{ in °C} \quad \text{(A-4-21M)}$$

where

T_{cr} = critical temperature in °F (°C)
R_n = nominal yielding strength at ambient temperature determined in accordance with the provisions in Section D2, kips (N)
R_u = required tensile strength at elevated temperature, determined using the load combination in Equation A-4-1 and greater than $0.01R_n$, kips (N)

User Note: Tensile rupture in the net section is not considered in this critical temperature calculation and ought to be considered using alternative methods.

(b) Design for Compression

The critical temperature of a compression member is permitted to be calculated as follows:

$$T_{cr} = 1580 - 0.814\left(\frac{L_c}{r}\right) - 1300\left(\frac{P_u}{P_n}\right) \text{ in °F} \quad \text{(A-4-22)}$$

$$T_{cr} = 858 - 0.455\left(\frac{L_c}{r}\right) - 722\left(\frac{P_u}{P_n}\right) \text{ in °C} \quad \text{(A-4-22M)}$$

where

L_c = effective length of member, in. (mm)
r = radius of gyration, in. (mm)
P_n = nominal compressive strength at ambient temperature determined in accordance with the provisions in Section E3, kips (N)
P_u = required compressive strength at elevated temperature, determined using the load combination in Equation A-4-1, kips (N)

(c) Design for Flexural Yielding

The critical temperature of a continuously braced beam not supporting a concrete slab is permitted to be calculated as follows:

$$T_{cr} = 816 - 306 \ln\left(\frac{M_u}{M_n}\right) \text{ in °F} \quad \text{(A-4-23)}$$

$$T_{cr} = 435 - 170 \ln\left(\frac{M_u}{M_n}\right) \text{ in °C} \quad \text{(A-4-23M)}$$

where

T_{cr} = critical temperature in °F (°C)

M_n = nominal flexural strength due to yielding at ambient temperature determined in accordance with the provisions in Section F2.1, kip-in. (N-mm)

M_u = required flexural strength at elevated temperature, determined using the load combination in Equation A-4-1, kip-in. and greater than $0.01M_n$ (N-mm)

User Note: Lateral-torsional buckling of beams is not considered in this critical temperature calculation and ought to be considered using alternative methods.

4.3. DESIGN BY QUALIFICATION TESTING

1. Qualification Standards

Structural members and components in steel buildings shall be qualified for the rating period in conformance with ASTM E119. Demonstration of compliance with these requirements using the procedures specified for steel construction in Section 5 of *Standard Calculation Methods for Structural Fire Protection* (ASCE/SEI/SFPE 29) is permitted. It is also permitted to demonstrate equivalency to such standard fire resistance ratings using the advanced analysis methods in Section 4.2 in combination with the fire exposure specified in ASTM E119 as the design-basis fire.

User Note: There are other standard fire exposures which are more severe than that prescribed in ASTM E119, for example the hydrocarbon pool fire scenario defined in ASTM E1529 (UL 1709). Fire resistance ratings developed on the basis of ASTM E119 are not directly substitutable for such more demanding conditions.

The generic steel assemblies described in Table A-4.3.1 shall be deemed to have the fire resistance ratings prescribed therein.

Table A-4.3.1[a]
Minimum Fire Protection and Fire Resistance Ratings of Steel Assemblies[e]

Assembly	Item Number	Fire Protection Material Used	Minimum Thickness (in.) of Insulating Material for Fire-Resistance Times (hr.)			
			4 hrs	3 hrs	2 hrs	1 hr
1. Steel columns and all of primary trusses	1-1.1	Carbonate, lightweight and sand-lightweight aggregate concrete, members 6 in. × 6 in. or greater (not including sandstone, granite and siliceous gravel).[a]	2-1/2	2	1-1/2	1
	1-1.2	Carbonate, lightweight and sand-lightweight aggregate concrete, members 8 in. × 8 in. or greater (not including sandstone, granite and siliceous gravel).[a]	2	1-1/2	1	1
	1-1.3	Carbonate, lightweight and sand-lightweight aggregate concrete, members 12 in. × 12 in. or greater (not including sandstone, granite and siliceous gravel).[a]	1-1/2	1	1	1
	1-1.4	Siliceous aggregate concrete and concrete excluded in Item 1-1.1, members 6 in. × 6 in. or greater.[a]	3	2	1-1/2	1
	1-1.5	Siliceous aggregate concrete and concrete excluded in Item 1-1.1, members 8 in.× 8 in.or greater.[a]	2-1/2	2	1	1
	1-1.6	Siliceous aggregate concrete and concrete excluded in Item 1-1.1, members 12 in.× 12 in or greater.[a]	2	1	1	1
	1-2.1	Clay or shale brick with brick and mortar fill.[a]	3-3/4	–	–	2-1/4

[a] ICC IBC-2018 *International Building Code,* International Code Council.

Table A-4.3.1[a] Minimum Fire Protection and Fire Resistance Ratings of Steel Assemblies[e] (continued)						
Assembly	Item Number	Fire Protection Material Used	Minimum Thickness (in.) of Insulating Material for Fire-Resistance Times (hr.)			
			4 hrs	3 hrs	2 hrs	1 hr
1. Steel columns and all of primary trusses						
	1-4.1	Cement plaster over metal lath wire tied to 3/4 in. cold-rolled vertical channels with 0.049 in. (No. 18 B.W. gage) wire ties spaced 3 to 6 in. on center. Plaster mixed 1:2.5 by volume, cement to sand.	–	–	2-1/2[b]	7/8

[a] ICC IBC-2018 *International Building Code,* International Code Council.

Table A-4.3.1[a]
Minimum Fire Protection and Fire Resistance Ratings of Steel Assemblies[e] (continued)

Assembly	Item Number	Fire Protection Material Used	Minimum Thickness (in.) of Insulating Material for Fire-Resistance Times (hr.)			
			4 hrs	3 hrs	2 hrs	1 hr
1. Steel columns and all of primary trusses	1-5.1	Vermiculite concrete, 1:4 mix by volume over paperbacked wire fabric lath wrapped directly around column with additional 2 × 2 in. 0.065 / 0.065 in. (No. 16/16 B.W. gage) wire fabric placed 3/4 in. from outer concrete surface. Wire fabric tied with 0.049 in. (No. 18 B.W. gage) wire spaced 6 in. on center for inner layer and 2 in. on center for outer layer.	2	–	–	–
	1-6.1	Perlite or vermiculite gypsum plaster over metal lath wrapped around column and furred 1-1/4 in. from column flanges. Sheets lapped at ends and tied at 6 in. intervals with 0.049 in. (No. 18 B.W. gage) tie wire. Plaster pushed through to flanges.	1-1/2	1	–	–
	1-6.2	Perlite or vermiculite gypsum plaster over self-furring metal lath wrapped directly around column, lapped 1 in. and tied at 6 in. intervals with 0.049 in. (No. 18 B.W. gage) wire.	1-3/4	1-3/8	1	–
	1-6.3	Perlite or vermiculite gypsum plaster on metal lath applied to 3/4 in. cold-rolled channels spaced 24 in. apart vertically and wrapped flatwise around column.	1-1/2	–	–	–

[a] ICC IBC-2018 *International Building Code,* International Code Council.

Table A-4.3.1[a]
Minimum Fire Protection and Fire Resistance Ratings of Steel Assemblies[e] (continued)

Assembly	Item Number	Fire Protection Material Used	Minimum Thickness (in.) of Insulating Material for Fire-Resistance Times (hr.)			
			4 hrs	3 hrs	2 hrs	1 hr
1. Steel columns and all of primary trusses	1-6.4	Perlite or vermiculite gypsum plaster over two layers of 1/2 in. plain full-length gypsum lath applied tight to column flanges. Lath wrapped with 1 in. hexagonal mesh of No. 20 gage wire and tied with doubled 0.035 in. diameter (No. 18 B.W. gage) wire ties spaced 23 in. on center. For three-coat work, the plaster mix for the second coat shall not exceed 100 pounds of gypsum to 2.5 cubic feet of aggregate for the 3-hour system.	2-1/2	2	–	–
	1-6.5	Perlite or vermiculite gypsum plaster over one layer of 1/2 in. plain full-length gypsum lath applied tight to column flanges. Lath tied with doubled 0.049 in. (No. 18 B.W. gage) wire ties spaced 23 in. on center and scratch coat wrapped with 1 in. hexagonal mesh 0.035 in. (No. 20 B.W. gage) wire fabric. For three-coat work, the plaster mix for the second coat shall not exceed 100 pounds of gypsum to 2.5 cubic feet of aggregate.	–	2	–	–

[a] ICC IBC-2018 *International Building Code,* International Code Council.

Table A-4.3.1[a]
Minimum Fire Protection and Fire Resistance Ratings of Steel Assemblies[e] (continued)

Assembly	Item Number	Fire Protection Material Used	Minimum Thickness (in.) of Insulating Material for Fire-Resistance Times (hr.)			
			4 hrs	3 hrs	2 hrs	1 hr
1. Steel columns and all of primary trusses	1-7.1	Multiple layers of 1/2 in. gypsum wallboard[c] adhesively[d] secured to column flanges and successive layers. Wallboard applied without horizontal joints. Corner edges of each layer staggered. Wallboard layer below outer layer secured to column with doubled 0.049 in. (No. 18 B.W. gage) steel wire ties spaced 15 in. on center. Exposed corners taped and treated.	–	–	2	1
	1-7.2	Three layers of 5/8 in. Type X gypsum wallboard.[c] First and second layer held in place by 1/8 in. dia. by 1-3/8 in. long ring shank nails with 5/16 in. dia. heads spaced 24 in. on center at corners. Middle layer also secured with metal straps at mid-height and 18 in. from each end, and by metal corner bead at each corner held by the metal straps. Third layer attached to corner bead with 1 in. long gypsum wallboard screws spaced 12 in. on center.	–	–	1-7/8	–

[a] ICC IBC-2018 *International Building Code,* International Code Council.

Table A-4.3.1[a]
Minimum Fire Protection and Fire Resistance Ratings of Steel Assemblies[e] (continued)

Assembly	Item Number	Fire Protection Material Used	Minimum Thickness (in.) of Insulating Material for Fire-Resistance Times (hr.)			
			4 hrs	3 hrs	2 hrs	1 hr
1. Steel columns and all of primary trusses	1-7.3	Three layers of 5/8 in. Type X gypsum wallboard,[c] each layer screw attached to 1-5/8 in. steel studs, 0.018 in. thick (No. 25 carbon sheet steel gage) at each corner of column. Middle layer also secured with 0.049 in. (No. 18 B.W. gage) double-strand steel wire ties, 24 in. on center. Screws are No. 6 by 1 in. spaced 24 in. on center for inner layer, No. 6 by 1-5/8 in. spaced 12 in. on center for middle layer and No. 8 by 2-1/4 in. spaced 12 in. on center for outer layer.	–	1-7/8	–	–

[a] ICC IBC-2018 *International Building Code,* International Code Council.

Table A-4.3.1[a] Minimum Fire Protection and Fire Resistance Ratings of Steel Assemblies[e] (continued)						
Assembly	Item Number	Fire Protection Material Used	Minimum Thickness (in.) of Insulating Material for Fire-Resistance Times (hr.)			
			4 hrs	3 hrs	2 hrs	1 hr
1. Steel columns and all of primary trusses	1-9.1	Minimum W8×35 wide flange steel column (w/d ≥ 0.75) with each web cavity filled even with the flange tip with normal weight carbonate or siliceous aggregate concrete (3,000 psi minimum compressive strength with 145 pcf ± 3 pcf unit weight). Reinforce the concrete in each web cavity with minimum No. 4 deformed reinforcing bar installed vertically and centered in the cavity, and secured to the column web with minimum No. 2 horizontal deformed reinforcing bar welded to the web every 18 in. on center vertically. As an alternative to the No. 4 rebar, 3/4 in. diameter by 3 in. long headed studs, spaced at 12 in. on center vertically, shall be welded on each side of the web midway between the column flanges.	–	–	–	See Note f
2. Webs or flanges of steel beams and girders	2.1-1	Carbonate, lightweight and sand-lightweight aggregate concrete (not including sandstone, granite and siliceous gravel) with 3 in. or finer metal mesh placed 1 in. from the finished surface anchored to the top flange and providing not less than 0.025 in.2 of steel area per foot in each direction.	2	1-1/2	1	1

[a] ICC IBC-2018 *International Building Code,* International Code Council.

Table A-4.3.1[a]
Minimum Fire Protection and Fire Resistance Ratings of Steel Assemblies[e] (continued)

Assembly	Item Number	Fire Protection Material Used	Minimum Thickness (in.) of Insulating Material for Fire-Resistance Times (hr.)			
			4 hrs	3 hrs	2 hrs	1 hr
2. Webs or flanges of steel beams and girders	2-1.2	Siliceous aggregate concrete and concrete excluded in Item 2-1.1 with 3 in. or finer metal mesh placed 1 in. from the finished surface anchored to the top flange and providing not less than 0.025 in.^{2} of steel area per foot in each direction.	2-1/2	2	1-1/2	1
	2-2.1	Cement plaster on metal lath attached to 3/4 in. cold-rolled channels with 0.04 in. (No. 18 B.W. gage) wire ties spaced 3 in. to 6 in. on center. Plaster mixed 1:2.5 by volume, cement to sand.	–	–	2-1/2[b]	7/8
	2-3.1	Vermiculite gypsum plaster on a metal lath cage, wire tied to 0.165 in. diameter (No. 8 B.W. gage) steel wire hangers wrapped around beam and spaced 16 in. on center. Metal lath ties spaced approximately 5 in. on center at cage sides and bottom.	–	7/8	–	–

[a] ICC IBC-2018 *International Building Code,* International Code Council.

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Table A-4.3.1[a]
Minimum Fire Protection and Fire Resistance Ratings of Steel Assemblies[e] (continued)

Assembly	Item Number	Fire Protection Material Used	Minimum Thickness (in.) of Insulating Material for Fire-Resistance Times (hr.)			
			4 hrs	3 hrs	2 hrs	1 hr
2. Webs or flanges of steel beams and girders	2-4.1	Two layers of 5/8 in. Type X gypsum wallboard[c] are attached to U-shaped brackets spaced 24 in. on center. 0.018 in. thick (No. 25 carbon sheet steel gage) 1-5/8 in. deep by 1 in. galvanized steel runner channels are first installed parallel to and on each side of the top beam flange to provide a 1/2 in. clearance to the flange. The channel runners are attached to steel deck or concrete floor construction with approved fasteners spaced 12 in. on center. U-shaped brackets are formed from members identical to the channel runners. At the bent portion of the U-shaped bracket, the flanges of the channel are cut out so that 1-5/8 in. deep corner channels can be inserted without attachment parallel to each side of the lower flange. As an alternative, 0.021 in. thick (No. 24 carbon sheet steel gage) 1 in. × 2 in. runner and corner angles shall be used in lieu of channels, and the web cutouts in the U-shaped brackets shall not be required. Each angle is attached to the bracket with 1/2-in.-long No. 8 self-drilling screws. The vertical legs of the U-shaped bracket are attached to the runners with one 1/2 in. long No. 8 self-drilling screw. The completed steel framing provides a 2-1/8 in. and 1-1/2 in. space between the inner layer of wallboard and the sides and bottom of the steel beam, respectively. The inner layer of wallboard is attached to the top runners and bottom corner channels or corner angles with 1-1/4 in.-long No. 6 self-drilling screws spaced 16 in. on center. The outer layer of wallboard is applied with 1-3/4 in.-long No. 6 self-drilling screws spaced 8 in. on center. The bottom corners are reinforced with metal corner beads.	–	–	1-1/4	–

[a] ICC IBC-2018 *International Building Code,* International Code Council.

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Table A-4.3.1[a]
Minimum Fire Protection and Fire Resistance Ratings of Steel Assemblies[e] (continued)

Assembly	Item Number	Fire Protection Material Used	Minimum Thickness (in.) of Insulating Material for Fire-Resistance Times (hr.)			
			4 hrs	3 hrs	2 hrs	1 hr
2. Webs or flanges of steel beams and girders	2-4.2	Three layers of 5/8 in. Type X gypsum wallboard[c] attached to a steel suspension system as described immediately above utilizing the 0.018 in. thick (No. 25 carbon sheet steel gage) 1 in. × 2 in. lower corner angles. The framing is located so that a 2-1/8 in. and 2 in. space is provided between the inner layer of wallboard and the sides and bottom of the beam, respectively. The first two layers of wallboard are attached as described immediately above. A layer of 0.035 in. thick (No. 20 B.W. gage) 1 in. hexagonal galvanized wire mesh is applied under the soffit of the middle layer and up the sides approximately 2 in. The mesh is held in position with the No. 6 1-5/8-in.-long screws installed in the vertical leg of the bottom corner angles. The outer layer of wallboard is attached with No. 6 2-1/4 in.-long screws spaced 8 in. on center. One screw is also installed at the mid-depth of the bracket in each layer. Bottom corners are finished as described above.	–	1-7/8	–	–

[a] Reentrant parts of protected members to be filled solidly.
[b] Two layers of equal thickness with a 3/4-in. airspace between.
[c] For all of the construction with gypsum wallboard, gypsum base for veneer plaster of the same size, thickness and core type is permitted to be substituted for gypsum wallboard, provided attachment is identical to that specified for the wallboard, the joints on the face layer are reinforced, and the entire surface is covered with not less than 1/16-inch gypsum veneer plaster.
[d] An approved adhesive qualified under ASTM E119.
[e] Generic fire-resistance ratings (those not designated as PROPRIETARY* in the listing) in GA 600 shall be accepted as if herein listed.

[a] ICC IBC-2018 *International Building Code,* International Code Council.

2. Structural Steel Assemblies

The provisions of this section contain procedures by which the standard fire-resistance ratings of structural steel assemblies are established by calculations. Use of these provisions is permitted in place of and/or as a supplement to published fire resistive assemblies based on ASTM E119. The installation of the fire protection material shall comply with the applicable requirements of the building code, the referenced approved assemblies, and manufacturer instructions.

The weight-to-heated-perimeter ratios (*W/D*) and area-to-heated-perimeter ratios (*A/P*) shall be determined in accordance with the definitions given in this section. As used in these sections, *W* is the average weight of a shape in pounds per linear foot and *A* is the area in square inches. The heated perimeter, *D* or *P*, is the inside perimeter of the fire-resistant material or exterior contour of the steel shape in inches, as defined for each type of member.

User Note: These procedures establish a basis for determining the fire resistance rating of steel construction assemblies as a function of the thickness of fire-resistant material, the weight, *W*, or area, *A*, and the applicable heated perimeter, *D* or *P*, of the fire protection material or structural steel member. The *W/D* and *A/P* ratios are equivalent and mutually convertible section properties that represent their thermal inertia. *W/D* has conventionally been used for open wide-flange shapes, while *A/P* has been used for closed hollow structural sections.

The heated perimeter, *D* or *P*, is a function of the configuration of the steel fire protection material installation, which can be in either a contour or box profile, together with the nature of the heat exposure on the steel member. The latter is typically characterized as either an all-around exposure of the steel shape, as for an interior column, or as a 3-sided exposure of a floor beam supporting a concrete floor. Tabulations of *W/D* and *A/P* values for these cases and for the standard rolled steel shapes are available from multiple sources, including AISC Design Guide 19 (a free download for members from www.aisc/org/dg) and other publications.

2.1 Steel Columns

The fire-resistance ratings of columns shall be based on the size of the member and the type of protection provided in accordance with this section.

The application of these procedures for noncomposite steel column assemblies shall be limited to designs in which the fire-resistant material is not designed to carry any of the load acting on the column.

Mechanical, electrical, and plumbing elements shall not be embedded in required fire-resistant materials, unless fire-endurance test results are available to establish the adequacy of the resulting condition.

User Note: The International Building Code requires fire resistance rated columns to be protected on all sides for the full column height, including connections with other structural members and protection continuity through any ceilings to the top of the column.

2.1.1 Gypsum Wallboard Protection

The fire resistance of columns with weight-to-heated perimeter ratios (W/D) less than or equal to 3.65 lb/ft/in. and protected with Type X gypsum wallboard is permitted to be determined from the following expression for a maximum column rating of 4-hours:

$$R = 130\left[\frac{h\left(\frac{W'}{D}\right)}{2}\right]^{0.75} \qquad \text{(A-4-24)[a],[b]}$$

$$R = 96\left[\frac{h\left(\frac{W'}{D}\right)}{2}\right]^{0.75} \qquad \text{(A-4-24M)[a]}$$

where

D = inside heated perimeter of the gypsum board, in. (mm)
R = fire resistance, minutes
W = nominal weight of steel shape, lb/ft (kg/m)
h = total nominal thickness of Type X gypsum wallboard, in. (mm)

and

$$\frac{W'}{D} = \frac{W}{D} + \frac{50h}{144} \qquad \text{(A-4-25)[a],[b]}$$

$$\frac{W'}{D} = \frac{W}{D} + 0.0008h \qquad \text{(A-4-25M)[a]}$$

For columns with weight-to-heated-perimeter ratios (W/D) greater than 3.65 lb/ft/in., the thickness of Type X gypsum wallboard required for specified fire-resistance ratings shall be the same as the thickness determined for W/D = 3.65 lb/ft/in.

User Note: This equation has been developed and long used for steel column fire protection with any Type X gypsum board. Since Type C gypsum board has demonstrated improved fire performance relative to Type X board, these provisions may also be conservatively applied to column protection with any Type C gypsum board. The supporting test data and accompanying gypsum board installation methods limit the computed fire resistance rating of the steel column to a maximum of 3-hours or 4-hours, as specified in the next section.

The gypsum board or gypsum panel products shall be installed and supported as required either in UL X526 for fire-resistance ratings of four hours or less, or in UL X528 for fire-resistance ratings of three hours or less.

User Note: The attachment of the Type X gypsum board protection for the steel columns must be done in accordance with the referenced UL assem-

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[a] ASCE/SEI/SFPE 29-05 *Standard Calculation Methods for Structural Fire Pro tection*
[b] ICC IBC-2018 *International Building Code,* International Code Council.

blies. UL X526 is applicable only when exterior steel covers are installed over the gypsum board. Otherwise, UL X528 describes the more general gypsum board installation.

2.1.2 Sprayed and Intumescent/Mastic Fire-Resistant Materials

The fire resistance of columns protected with sprayed or intumescent/mastic fire-resistant coatings shall be determined on the basis of standard fire-resistance rated assemblies, any associated computations and limits as provided in the applicable rated assemblies.

The fire resistance of wide-flange columns protected with sprayed fire-resistant materials is permitted to be determined as:

$$R=\left[C_1\left(\frac{W}{D}\right)+C_2\right]h \qquad \text{(A-4-26)}^{[a],[b]}$$

$$R=\left[C_3\left(\frac{W}{D}\right)+C_4\right]h \qquad \text{(A-4-26M)}^{[a]}$$

where

R = fire resistance, minutes
h = thickness of sprayed fire-resistant material, in.
D = heated perimeter of the column, in.
C_1, C_2, C_3, and C_4 = material-dependent constants prescribed in specified rated assembly.
W = weight of columns, pounds per linear foot

The material dependent constants, C_1, C_2, C_3, and C_4 shall be determined for specific fire-resistant materials on the basis of standard fire endurance tests. The computational usage for each correlation, protection product and its material-dependent constants shall be limited to the range of their underlying fire test basis reflected in the selected rated assembly.

User Note: The fire resistance rated steel column assemblies, published by UL and by other test laboratories, will often include such interpolation equations and specific constants that depend on the particular fire protection product. The applicability limits of each given design correlation relative to the column assembly, sprayed fire-resistant protection product, *W/D*, rating duration, minimum required thickness, and the like must be followed to remain within the range of the existing fire test data range.

The fire resistance of HSS columns protected with sprayed fire-resistant materials is permitted to be determined from empirical correlations similar to Equation A-4-25 expressed in terms of *A/P* values, wherein *A* is the area in in.^{2} (mm^2) and *P* is the heated perimeter. The applicability limits specified in the rated column assembly for each correlation and its material-dependent constants shall be followed.

User Note: *A/P* is a directly convertible and equivalent steel section property to *W/D* which has traditionally been used in fire resistive compu-

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[a] ASCE/SEI/SFPE 29-05 *Standard Calculation Methods for Structural Fire Protection*
[b] ICC IBC-2018 *International Building Code,* International Code Council.

tations for HSS sections. Similar to *W/D* for open wide flange shapes, tabulation of *A/P* values for standard closed shapes with contour and box protection applications are available from multiple sources, including AISC and the published literature. The applicability limits of each given design correlation relative to the column assembly, sprayed fire-resistant protection product, *A/P*, rating duration, minimum required thickness, and the like must be followed to remain within the range of the existing fire test result range .

2.1.3 Noncomposite Columns Encased in Concrete

The fire resistance of noncomposite columns fully encased within concrete protection is permitted to be determined from the following expression:

$$R = R_o\left(1+0.03m\right) \qquad \text{(A-4-27)}^{[a],[b]}$$

where

$$R_o = 10\left(\frac{W}{D}\right)^{0.7} + 17\left(\frac{h^{1.6}}{k_c^{\ 0.2}}\right)\times\left\{1+26\left[\frac{H}{p_c c_c h\left(L+h\right)}\right]^{0.8}\right\} \qquad \text{(A-4-28)}^{[a],[b]}$$

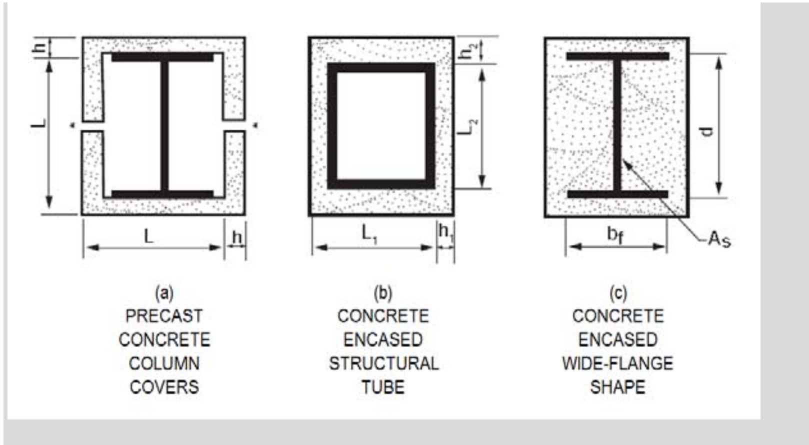
$$R_o = 73\left(\frac{W}{D}\right)^{0.7} + 0.162\frac{h^{1.6}}{k_c^{0.2}}\left\{1+31{,}000\left[\frac{H}{p_c c_c h(L+h)}\right]^{0.8}\right\} \qquad \text{(A-4-28M)}^{[a]}$$

R = fire endurance at equilibrium moisture conditions, minutes
R_o = fire endurance at zero moisture content, minutes
m = equilibrium moisture content of the concrete by volume, percent
W = average weight of the column, lb/ft (kg/m)
D = heated perimeter of the column, in. (mm)
h = thickness of the concrete cover, measured between the exposed concrete and nearest outer surface of the encased steel column section, in. (mm)
k_c = ambient temperature thermal conductivity of the concrete, Btu/hr ft °F. (W/m K)
H = ambient temperature thermal capacity of the steel column, Btu/ ft °F (W/kJ m K)
= $0.11W$ ($0.46W$)
p_c = concrete density, lb/ft^3 (kg/m^3)
c_c = ambient temperature specific heat of concrete, Btu/lb °F (kJ/kg K)
L = interior dimension of one side of a square concrete box protection, in. (mm)

When the inside perimeter of the concrete protection is not square, L shall be taken as the average of its two rectangular side lengths (L_1 and L_2). If the thickness of the concrete cover is not constant, h shall be taken as the average of h_1 and h_2.

User Note: The variables in these equations are illustrated in the figure

[a] ASCE/SEI/SFPE 29-05 *Standard Calculation Methods for Structural Fire Pro tection*
[b] ICC IBC-2018 *International Building Code,* International Code Council.



For wide-flange columns completely encased in concrete with all reentrant spaces filled, the thermal capacity of the concrete within the reentrant spaces is permitted to be added to the ambient thermal capacity of the steel column, as follows:

$$H = 0.11W + \left(\frac{p_c c_c}{144}\right)\left(b_f d - A_s\right) \qquad \text{(A-4-29)}^{[a],[b]}$$

$$H = 0.46W + \left(\frac{p_c c_c}{1{,}000{,}000}\right)\left(b_f d - A_s\right) \qquad \text{(A-4-29M)}^{[a]}$$

where:

b_f = flange width of the column, in. (mm)

d =depth of the column, in. (mm)

A_s = area of the steel column, in.^{2} (mm^2)

User Note: It is conservative to neglect this additional concrete term in the column fire resistance calculation.

In the absence of more specific data for the ambient properties of the concrete encasement, it is permitted to use the values provided in Table A-4.3.2.

Table A-4.3.2[c]

Ambient Properties of Concrete Encasement for Steel Column Fire Resistance

Property	Normal Weight Concrete	Light Weight Concrete
Thermal conductivity, k_c	0.95 Btu/hr ·ft ·°F (1.64 W/m K)	0.35 Btu/hr · ft ·°F (0.61 W/m K)
Specific heat, c_c	0.20 Btu/lb °F (840 J/kg K)	0.20 Btu/lb °F (840 J/kg K)

[a] ASCE/SEI/SFPE 29-05 *Standard Calculation Methods for Structural Fire Pro tection*

[b] ICC IBC-2018 *International Building Code,* International Code Council.

[c] ICC IBC-2018 *International Building Code,* International Code Council.

Density, p_c	145 lb/ft^3 (2300 kg/m^3)	110 lb/ft^3 (1800 kg/m^3)
Equilibrium (free) moisture content (m) by volume	4%	5%

User Note: The estimated free moisture content of concrete given in Table A-4.3.2 may not be appropriate for all conditions, particularly for older concrete that has already been in service for a longer time. For these and similar situations of uncertainty, it is conservative to not rely on this beneficial effect of the free moisture and to assume the concrete is completely dry with m=0 for fire resistance of R_o.

2.1.4 Noncomposite Columns Encased in Masonry Units of Concrete or Clay

The fire resistance of noncomposite columns protected by encasement with concrete masonry units or with clay masonry units is permitted to be determined from the following expression:

$$R = 0.17\left(\frac{W}{D}\right)^{0.7} + \left[0.285\left(\frac{T_e^{1.6}}{K^{0.2}}\right)\right]\left\{1.0 + 42.7\left[\frac{\left(A_s/d_m T_e\right)}{\left(0.25p + T_e\right)}\right]^{0.8}\right\} \qquad \text{(A-4-30)}$$ [a]

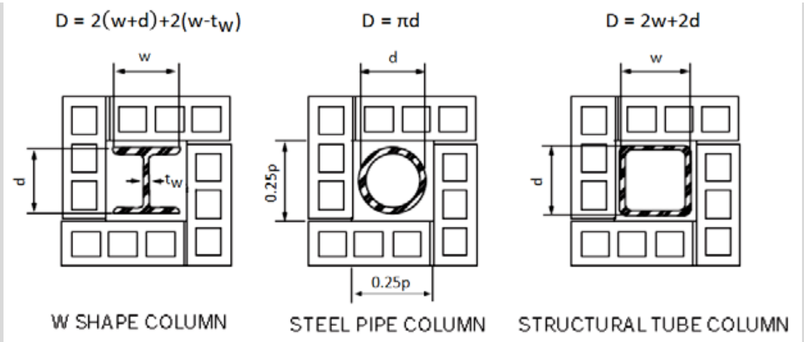
$$R = 1.22\left(\frac{W}{D}\right)^{0.7} + \left[0.0027\left(\frac{T_e^{1.6}}{K^{0.2}}\right)\right]\left\{1.0 + 1249\left[\frac{\left(\frac{A_s}{d_m T_e}\right)}{(0.25p + T_e)}\right]^{0.8}\right\} \qquad \text{(A-4-30M)}$$

where

R = fire-resistance rating of column assembly, hours
W = average weight of column, lb/ft (kg/m)
D = heated perimeter of column, in. (mm)
T_e = equivalent thickness of concrete or clay masonry unit, in accordance with ACI 216.1, in. (mm)
K = thermal conductivity of concrete or clay masonry unit, Btu/hr · ft · °F (see Table A-4.3.3).
A_s = cross-sectional area of column, in.^{2} (mm^2)
d_m = density of the concrete or clay masonry unit, lb/ft^3 (kg/m^3) p = inner perimeter of concrete or clay masonry protection, in. (mm)

The thermal conductivity values given in Table A-4.3.3 as a function of the concrete or clay masonry unit density is permitted for use with this encasement protection formulation.

User Note: Equation A-4-30 is derived from Equation A-4-27 assuming m = 0, c_c = 0.2 Btu/lb °F, h = T_e, and L = p/4. The following cross-sections illustrate three different configurations for concrete masonry units or clay masonry unit encasement of steel columns, along with the applicable fire protection design variables.



d = depth of a wide flange column, outside diameter of pipe column, or outside dimension of hollow structural section column, in. (mm)
t_w = thickness of web of wide flange column, in. (mm)
w = width of flange of wide flange or hollow structural section, in. (mm)

Table A-4.3.3[a]
Thermal Conductivity of Masonry Units for Steel Column Encasement

Unit Density, d_m, lb/ft^3 (kg/m^3)	Unit Thermal Conductivity K, Btu/hr ft °F (W/m K)
Concrete Masonry Units	
80 (1280)	0.207 (0.358)
85 (1360)	0.228 (0.395)
90 (1440)	0.252 (0.436)
95 (1520)	0.278 (0.481)
100 (1600)	0.308 (0.533)
105 (1680)	0.340 (0.589)
110 (1760)	0.376 (0.651)
115 (1840)	0.416 (0.720)
120 (1920)	0.459 (0.795)
125 (2000)	0.508 (0.879)
130 (2080)	0.561 (0.971)
135 (2160)	0.620 (1.07)
140 (2240)	0.685 (1.19)
145 (2320)	0.758 (1.31)
150 (2400)	0.837 (1.45)
Clay Masonry Units	
120 (1920)	1.25 (2.16)
130 (2080)	2.25 (3.89)

2.2 Composite Steel-Concrete Columns

The fire resistance rating of columns acting compositely with concrete (concrete-filled or encased) is permitted to be based on the size of the composite member and concrete protection in accordance with this section.

2.2.1 Concrete-Filled Columns

The fire resistance rating of hollow structural section (HSS) columns filled with unreinforced normal weight concrete, steel-fiber-reinforced normal

[a] ICC IBC-2018 *International Building Code,* International Code Council.

weight concrete or bar-reinforced normal weight concrete is permitted to be determined in accordance with the following expressions:

(A-4-31)[a]

$$R = \frac{0.58a(f_c' + 2.9)D^2\left(\frac{D}{C}\right)^{0.5}}{L_c - 3.28}$$

(A-4-31M)[a]

$$R = \frac{a(f_c' + 20)D^2\left(\frac{D}{C}\right)^{0.5}}{[60(L_c - 1000)]}$$

R = fire resistance rating in hours
a = constant determined from Table A-4.3.4
f'_c = 28-day compressive strength of concrete, ksi(MPa)
L_c = column effective length, ft (mm)
D = outside diameter for circular columns, in. (mm)
= outside dimension for square columns, in. (mm)
= least outside dimension for rectangular columns, in. (mm)
C = compressive force due to unfactored dead load and live load, kips (kN)

The application of these equations shall be limited by all of the following conditions:

1. The required fire resistance rating R shall be less than or equal to the limits specified in Tables A-4.3.5 or A-4.3.5M.
2. The specified compressive strength of concrete, f'_c, the column effective length, L_c, the dimension D, the concrete reinforcement ratio, and the thickness of the concrete cover shall be within the limits specified in Tables A-4.3.5 or A-4.3.5M.
3. C shall not exceed the design strength of the concrete or the reinforced concrete core determined in accordance with this Specification.
4. Two minimum 1/2 in.(12.7 mm) diameter holes shall be placed opposite each other at the top and bottom of the column and at maximum 12-ft on center spacing along the column height. Each set of vent holes should be rotated 90° relative to the adjacent set of holes to relieve steam pressure.

User Note: Concrete-filled hollow structural sections (HSS) can effectively sustain load during a fire exposure without benefit of any external protection for the steel HSS. The concrete infill mass provides both an increased capacity for absorbing the heat caused by the fire and loadbearing strength to thereby extend the column fire resistance duration. Research conducted at the National Research Council of Canada has provided a basis for establishing an empirical equation to predict the standard fire resistance of concrete-filled round and square HSS section for commonly used story heights and steel sections. This empirical equation was derived from and can only be used within the allowable range of design variables, as given, and is not applicable to lightweight concrete infill.

[a] ASCE/SEI/SFPE 29-05 *Standard Calculation Methods for Structural Fire Protection*

The fire performance of a concrete-filled HSS column is improved when heat absorption occurs as the moisture in the concrete is converted to steam. The heat absorbed during this phase change is significant, however the resulting steam must be released to prevent the adverse effects of an internal pressure build-up within the HSS column. Thus, vent holes must be provided in the steel section, as indicated in the given limitation #4.

Table A-4.3.4 Values of Constant *a* for Normal Weight Concrete

Aggregate Type	Concrete Fill Type	Reinf. Ratio (%)	*a*	
			Circular Columns	Sq. or Rect. Columns
siliceous	unreinforced	NA	0.070	0.060
siliceous	steel-fiber-reinforced	2 %	0.075	0.065
siliceous	steel-bar-reinforced	1.5 – 3	0.080	0.070
		3 – 5	0.085	0.070
carbonate	unreinforced	NA	0.080	0.070
carbonate	steel-fiber-reinforced	2	0.085	0.075
carbonate	steel-bar-reinforced	1.5 - 3	0.090	0.080
		3 – 5	0.095	0.085

Table A-4.3.5 Limits for the use of Equation A-4.25 Parameters

Parameter	Concrete Fill Type		
	Unreinforced	steel-fiber-reinforced	steel-bar reinforced
R (hours)	≤ 2	≤ 3	≤ 3
fc’ (ksi)	2.9 – 5.8	2.9 – 8.0	2.9 – 8.0
L_c (ft)	6.5 – 13.0	6.5 – 15.0	6.5 – 15.0
D (round) (in)	5.5 – 16.0	5.5 – 16.0	6.5 – 16.0
D (sq. or rect.) (in)	5.5 – 12.0	4.0 – 12.0	7.0 – 12.0
Reinf. (%)	NA	2% of concrete mix by mass	1.5 – 5% of section area
Concrete cover (in)	NA	NA	≥ 1.0

Table A-4.3.5M. Limits for the use of Equation A-4.25MParameters

Parameter	Concrete Fill Type		
	unreinforced	steel-fiber-reinforced	steel-bar-reinforced
R (hours)	≤ 2	≤ 3	≤ 3
fc’ (MPa)	20 – 40	20-55	20-55
L_c (mm)	2000 – 4000	2000 - 4500	2000 - 4500
D (round) (mm)	140 – 410	140 - 410	165 - 410
D (sq. or rect.) (mm)	140 – 305	102 - 305	175 - 305
Reinf. (%)	NA	2% of concrete mix by mass	1.5 – 5% of section area
Concrete cover (mm)	NA	NA	≥ 25

2.2.2 Composite Columns Encased in Concrete

The fire resistance of composite columns fully encased within normal weight or lightweight concrete and with no unfilled spaces is permitted to be determined as the lesser of Equation A-4-30 and the values in Table A-4.3.6.

Table A-4.3.6 Minimum size and concrete cover limits for fire resistance of composite steel columns encased in concrete with no unfilled spaces

Fire Resistance Rating, hrs	Minimum Concrete Cover, *h*, in. (mm)	Minimum Column Outside Dimension, in. (mm)
1	1 (25)	8 (200)
2	2 (50)	10 (250)
3	2 (50)	12 (300)
4	2 (50)	14 (350)

User Note: The fire resistance ratings and requirements in Table A-4.3.7 were directly adapted from the ACI 216.1 provisions for conventional bar-reinforced concrete columns. Substitution of an embedded structural steel shape for steel bar reinforcement should not reduce the fire resistance of the loadbearing concrete parts of column, and the R computed for the same but assumed non-composite steel column accordingly verifies the fire resistance of the loadbearing steel shape. The concrete cover, *h*, is defined identical to that used for non-composite steel columns encased in concrete.

2.3 I-Shaped Beams and Girders

The fire-resistance ratings of beams and girders shall be based upon the size of the element and the type of protection provided in accordance with this section.

These procedures establish a basis for determining resistance of structural steel beams and girders that differ in size from that specified in approved fire-resistance-rated assemblies as a function of the thickness of fire-resistant material and the weight (*W*) and heated perimeter (*D*) of the beam or girder.

The beams provided in approved fire-resistance-rated assemblies shall be considered to be the minimum permissible size. Other beam or girder shapes is permitted to be substituted provided that the weight-to-heated-perimeter ratio (*W/D*) of the substitute beam is equal to or greater than that of the minimum beam specified in the approved assembly.

User Note: In the past, the substitution of larger beams for the minimum required sizes has been permitted based upon the thickness of web and flange elements, *W/D* ratio, or the beam size designation. Extensive fire research has shown that the heat transfer to a protected steel beam or girder is actually a direct function of the *W/D* ratio. As a result, beam substitutions should be more directly based upon *W/D* ratios. The significance of the thickness of web and flange elements and beam size is inherently included in the determination of *W/D* ratios.
It is acceptable and conservative to protect a larger steel beam or girder, which has a greater *W/D* value than the *W/D* of the minimum member size

specified in an approved assembly, with the thickness of fire protection material required for the minimum member size.

2.3.1 Sprayed and Intumescent/Mastic Fire-Resistant Materials

The provisions in this section apply to beams and girders protected with sprayed or intumescent/mastic fire-resistant materials.

Larger or smaller beam and girder shapes protected with sprayed fire-resistant materials are permitted to be substituted for beams specified in approved unrestrained or restrained fire-resistance-rated assemblies, provided that the thickness of the fire-resistant material is adjusted in accordance with the following expression:

$$h_2 = h_1 [(W_1 / D_1) + 0.60] / [(W_2 / D_2) + 0.60] \qquad \text{(A-4-32)}$$ [a],[b]

$$h_2 = h_1[(W_1/D_1) + 0.036]/[(W_2/D_2) + 0.036] \qquad \text{(A-4-32M)}$$ [a]

where:
h = thickness of sprayed fire-resistant material, in. (mm)
W = weight of the beam or girder, lb/ft (kg/m)
D = heated perimeter of the beam, in. (mm)

Subscript 1 refers to the substitute beam or girder and the required thickness of fire-resistant material.
Subscript 2 refers to the beam and fire-resistant material thickness in the approved assembly.

The use of this Equation is limited to the following conditions:

1. The weight-to-heated-perimeter ratio for the substitute beam or girder (W_1/D_1) shall be not less than 0.37 (customary units) or 0.022 (SI units).
2. The thickness of fire protection materials calculated for the substitute beam or girder (T_1) shall be not less than 3/8 in. (10 mm).
3. The unrestrained or restrained beam rating shall be not less than 1 hour.
4. Where used to adjust the material thickness for a restrained beam, the use of this procedure is limited to sections classified as compact.

User Note: This substitution equation based on W/D for beams protected with spray-applied fire resistive materials was developed by UL with the given limitations. The minimum *W/D* ratio of 0.37 prevents the use of this equation for determining the fire resistance of very small shapes that have not been tested. The 3/8-in. (10 mm) minimum thickness of protection is a practical application limit based upon the most commonly used spray-applied fire protection materials.

The fire resistance of beams and girders protected with intumescent or mastic fire-resistant coatings shall be determined on the basis of standard

[a] ASCE/SEI/SFPE 29-05 *Standard Calculation Methods for Structural Fire Protection*.
[b] ICC IBC-2018 *International Building Code,* International Code Council.

fire-resistance rated assemblies, and associated computations and limits as provided in the applicable rated assemblies.

2.4 Trusses

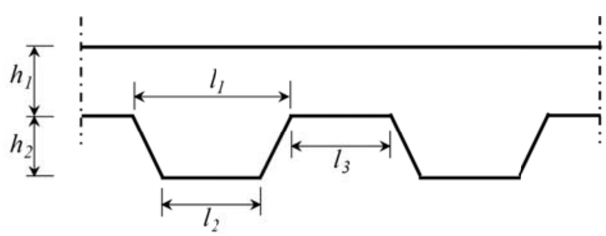
The fire resistance of trusses with members individually protected by fire-resistant materials applied onto each of the individual truss elements is permitted to be determined for each member in accordance with the Appendix 4, Section 4.3.1. The protection thickness of truss elements that can be simultaneously exposed to fire on all sides shall be determined for the same weight-to-heated perimeter ratio (*W/D*) as columns. The protection thickness of truss elements that directly support floor or roof assembly is permitted to be determined for the same weight-to-heated-perimeter ratio (*W/D*) as for beams and girders.

User Note: For trusses, application of the column fire resistance equation is more technically correct than the beam equation, since truss members are predominantly axially loaded and will require larger protection thicknesses than beams. Also, most truss elements can be exposed to fire on all four sides simultaneously. As a result, the heated perimeter and protection thickness of most truss members should be determined in the same manner as for columns. However, an exception is included for top chord elements that directly support floor or roof construction. The heated perimeter and protection thickness of such elements may be determined in the same manner as for beams and girders, or they may be conservatively determined in the same manner as for columns.

2.5 Concrete Floor Slabs on Steel Deck

For composite concrete floor slabs on trapezoidal steel decking wherein the upper width of the deck rib is equal to or greater than its bottom rib width, the fire resistance rating, based on the thermal insulation criterion for the unexposed surface temperature, shall be permitted to be calculated using the following equation:

$$R = a_0 + a_1 h_1 + a_2 h_2 + a_3 l_2 + a_4 l_3 + a_5 m + a_6 h_1^2 + a_7 h_1 h_2 + a_8 h_1 l_2 + a_9 h_1 l_3 + a_{10} h_1 m + a_{11} h_2 l_2 + a_{12} h_2 l_3 + a_{13} h_2 m + a_{14} l_2 l_3 + a_{15} l_2 m + a_{16} l_3 m \qquad \text{(A-4-33)}$$



where

R = fire resistance rating in minutes
h_1 = concrete slab thickness above steel deck, in (mm)
h_2 = depth of steel deck, in (mm)
l_1 = largest upper width of deck rib, in (mm)
l_2 = bottom width of deck rib, in (mm)
l_3 = width of deck upper flange, in (mm)

m = moisture content of the concrete slab. Range of applicability is between 0% (0.0) and 10% (0.1)

The coefficients a_0 to a_{16} are shown in Table A-4.3.7.

TABLE A-4.3.7 Coefficients a_0 to a_{16} for use with Equation A-4-33

Coefficient	Coefficient Value	
	Normal-weight concrete	Lightweight concrete
a_0	38.6 min	68.7 min
a_1	-5.08 min/in (-0.2 min/mm)	-36.58 min/in (-1.44 min/mm)
a_2	−1.45 min/in (−0.057 min/mm)	-2.79 min/in (-0.11 min/mm)
a_3	−3.30 min/in (−0.13 min/mm)	−12.70 min/in (−0.5 min/mm)
a_4	−2.08 min/in (−0.082 min/mm)	20.07 min/in (0.79 min/mm)
a_5	-118.1 min	-784.2 min
a_6	4.06 min/in^2 (0.0063 min/mm^2)	8.84 min/in^2 (0.0137 min/mm^2)
a_7	1.48 min/in^2 (0.0023 min/mm^2)	3.61 min/in^2 (0.0056 min/mm^2)
a_8	1.87 min/in^2 (0.0029 min/mm^2)	3.68 min/in^2 (0.0057 min/mm^2)
a_9	0	-2.39 min/in^2 (-0.0037 min/mm^2)
a_{10}	263.1 min/in (10.36 min/mm)	444.5 min/in (17.5 min/mm)
a_{11}	1.16 min/in^2 (0.0018 min/mm^2)	2.06 min/in^2 (0.0032 min/mm^2)
a_{12}	0	-3.42 min/in^2 (-0.0053 min/mm^2)
a_{13}	0	91.44 min/in (3.6 min/mm)
a_{14}	-0.65 min/in^2 (-0.001 min/mm^2)	-0.97 min/in^2 (-0.0015 min/mm^2)
a_{15}	0	42.42 min/in (1.67 min/mm)
a_{16}	0	-66.04 min/in (-2.6 min/mm)

User Note: If moisture content values are not available, m = 4% and 5% can be used for normal-weight concrete and lightweight concrete, respectively, consistent with Annex D of Eurocode 4. Dry conditions (m = 0%) will yield the most conservative fire resistance rating.

2.6 Composite Plate Shear Walls

For unprotected composite plate shear walls meeting the requirements of Chapter I and Section 4.3.2.6, the fire resistance rating is permitted to be determined in accordance with Equation A-4-34.

$$R = \left[-18.5\left(\frac{P_u}{P_n}\right)^{\left(0.24-\frac{L/t_{sc}}{230}\right)} + 15\right]\left(\frac{1.9t_{sc}}{8} - 1\right) \quad \text{(A-4-34)}$$

$$R = \left[-18.5\left(\frac{P_u}{P_n}\right)^{\left(0.24-\frac{L/t_{sc}}{230}\right)} + 15\right]\left(\frac{1.9t_{sc}}{200} - 1\right) \quad \text{(A-4-34M)}$$

where R is the fire rating in hours, P_u is the applied axial load in kips (kN), and L, t_{sc}, and P_n are as defined in Chapter I.

The use of Equation A-4-34 shall be limited to walls satisfying all the following conditions:

1. Wall slenderness ratio (L/t_{sc}) is less than or equal to 20
2. Axial load ratio (P_u/P_n) is less than or equal to 0.2
3. Wall thickness, t_{sc}, is between 8 in. and 24 in. (200 mm and 600 mm)

3. 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 forces and accommodating deformations caused by thermal expansion throughout the range of anticipated elevated temperatures. Cast-in-place or prefabricated concrete floor or roof construction secured to steel framing members, and individual steel beams and girders that are welded or bolted to integral framing members shall be considered restrained construction.

4. 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 effects of elevated temperatures.

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 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. Load testing in accordance with this appendix applies to static vertical gravity load effects.

The Appendix is organized as follows:

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, when specified in the contract documents by the engineer of record (EOR).

5.2. MATERIAL PROPERTIES

For evaluations in accordance with this appendix, steel grades other than those listed in Section A3.1 are permitted.

1. Determination of Required Tests

The EOR shall determine the specific tests that are required from Sections 5.2.2 through 5.2.6 and specify the locations where they are required. The use of applicable project records is permitted to reduce or eliminate the need for testing.

2. Tensile Properties

Tensile properties of members shall be established for use 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. Certified material 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, are permitted for this purpose. Otherwise, tensile tests shall be conducted in accordance with ASTM A370 from samples taken 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. Results from certified material test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM procedures are 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 shall 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 EOR 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 *Structural Welding Code—Steel*, AWS D1.1/D1.1M, are not met, the EOR shall determine if remedial actions are required.

6. Bolts and Rivets

Representative samples of bolts shall be visually inspected to determine markings and classifications. Where it is not possible to classify bolts by visual inspection, representative samples shall be taken and tested to determine tensile strength in accordance with ASTM F606/F606M and the bolt classified accordingly. Alternatively, the assumption that the bolts are ASTM A307 is 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, it is permitted to determine such dimensions from applicable project design or fabrication documents 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 and Appendix 5 of this Specification.

2a. Rivets

The design tensile or shear strength, ϕR_n, and the allowable tensile or shear strength, R_n/Ω, of a driven rivet shall be determined according Section J3.6, and driven rivets under combined tension and shear shall satisfy the requirements of Section J3.7,

where

A_b = nominal body area of undriven rivet, in.^{2} (mm^2)

F_{nt} = nominal tensile strength of the driven rivet from Table A-5.3.1, ksi (MPa)

F_{nv} = nominal shear strength of the driven rivet from Table A-5.3.1, ksi (MPa)

Table A-5.3.1
Design Strength of Rivets

Description of Rivet	Nominal Tensile Strength, ksi (MPa) [a]	Nominal Shear Strength, ksi (MPa) [b]
A502, Grade 1, hot-driven rivets	45 (310)	25 (170)

[a] Static loading only.
[b] Refer to Note [b] of Table J3.2.

3. Serviceability Evaluation

Where required, the deformations at service loads shall be calculated and reported.

5.4. EVALUATION BY LOAD TESTS

1. General Requirements

This section applies only to static vertical gravity loads applied to existing roofs or floors.

Where load tests are used, the EOR shall first analyze the structure, prepare a testing plan, and develop a written procedure for the test. The plan shall consider catastrophic collapse and/or excessive levels of permanent deformation, as defined by the EOR, and shall include procedures to preclude either occurrence during testing.

2. 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 EOR's plan. The structure shall be visually inspected for signs of distress or imminent failure at each load level. Measures shall be taken to prevent collapse 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. For roof structures, L_r, S or R shall be substituted for L,

where
L_r = nominal roof live load
R = nominal load due to rainwater or snow, exclusive of the ponding contribution
S = nominal snow load

More severe load combinations shall be used where required by the applicable building codes.

Periodic unloading is permitted 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% above that at the beginning of the holding period. It is permissible to repeat the test loading 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.

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 condition shall be selected.

3. Serviceability Evaluation

When load tests are prescribed, the structure shall be loaded incrementally to the service load level. The service test load shall be held for a period of one hour, and deformations shall be recorded at the beginning and at the end of the one-hour holding period.

5.5. EVALUATION REPORT

After the evaluation of an existing structure has been completed, the EOR 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 documents, material test reports, and auxiliary material testing shall also be reported. The report shall indicate whether the structure, including all members and connections, can withstand the load effects.

APPENDIX 6

MEMBER STABILITY BRACING

This appendix addresses the minimum strength and stiffness necessary for bracing to develop the required strength of a column, beam, or beam-column. The appendix is organized as follows:

6.1. General Provisions
6.2. Column Bracing
6.3. Beam Bracing
6.4. Beam-Column Bracing

User Note: Stability requirements for lateral force-resisting systems are provided in Chapter C. The provisions in this appendix apply to bracing that is not generally included in the analysis model of the overall structure, but is provided to stabilize individual columns, beams and beam-columns. Guidance for applying these provisions to stabilize trusses is provided in the Commentary.

6.1. GENERAL PROVISIONS

Bracing systems shall have the strength and stiffness specified in this Appendix, as applicable. Where such a system braces more than one member, the strength and stiffness of the bracing shall be based on the sum of the required strengths of all members being braced and consider the flexibility of all components in the system. The evaluation of the stiffness furnished by the bracing shall include the effects of connections and anchoring details.

User Note: More detailed analyses for bracing strength and stiffness are presented in the Commentary.

A panel brace (formerly referred to as a relative brace) controls the angular deviation of a segment of the braced member between braced points (that is, the lateral displacement of one end of the segment relative to the other). A point brace (formerly referred to as a nodal brace) controls the movement at the braced point without direct interaction with adjacent braced points. A continuous bracing system consists of bracing that is attached along the entire member length.

The available strength and stiffness of the bracing members and connections shall equal or exceed the required strength and stiffness, respectively, unless analysis indicates that smaller values are justified.

Columns, beams and beam-columns with end and intermediate braced points designed to meet the requirements in Sections 6.2, 6.3 and 6.4, as applicable, are permitted to be designed based on lengths L_c and L_b, as defined in Chapters E and F, taken equal to the distance between the braced points.

In lieu of the requirements of Sections 6.2, 6.3 and 6.4,

(a) The required brace strength and stiffness can be obtained using a second-order analysis that satisfies the provisions of Chapter C or Appendix 1, as appropriate, and includes brace points displaced from their nominal locations in a pattern that provides for the greatest demand on the bracing.

(b) The required bracing stiffness can be obtained as $2/\phi$ (LRFD) or 2Ω (ASD) times the ideal bracing stiffness determined from a buckling analysis. The required brace strength can be determined using the provisions of Sections 6.2, 6.3 and 6.4, as applicable.

(c) For either of the above analysis methods, members with end or intermediate braced points meeting these requirements may be designed based on effective lengths, L_c and L_b, taken less than the distance between braced points.

User Note: The stability bracing requirements in Sections 6.2, 6.3 and 6.4 are based on buckling analysis models involving idealizations of common bracing conditions. Computational analysis methods may be used for greater generality, accuracy and efficiency for more complex bracing conditions. The Commentary to Section 6.1 provides guidance on these considerations.

6.2. COLUMN BRACING

It is permitted to laterally brace an individual column at end and intermediate points along its length using either panel or point bracing.

User Note: This section provides requirements only for lateral bracing. Column lateral bracing is assumed to be located at the shear center of the column. When lateral bracing does not prevent twist, the column is susceptible to torsional buckling, as addressed in Section E4. When the lateral bracing is offset from the shear center, the column is susceptible to constrained-axis torsional buckling, which is addressed in the commentary to Section E4.

1. Panel Bracing

The panel bracing system shall have the strength and stiffness specified in this section. The connection of the bracing system to the column shall have the strength specified in Section 6.2.2 for a point brace at that location.

User Note: If the stiffness of the connection to the panel bracing system is comparable to the stiffness of the panel bracing system itself, the panel bracing system and its connection to the column function as a panel and point bracing system arranged in series. Such cases may be evaluated using the alternative analysis methods listed in Section 6.1.

In the direction perpendicular to the longitudinal axis of the column, the required shear strength of the bracing system is:

$$V_{br} = 0.005P_r \tag{A-6-1}$$

and, the required shear stiffness of the bracing system is:

$$\beta_{br} = \frac{1}{\phi}\left(\frac{2P_r}{L_{br}}\right) \text{ (LRFD)} \tag{A-6-2a}$$

$$\beta_{br} = \Omega\left(\frac{2P_r}{L_{br}}\right) \text{ (ASD)} \tag{A-6-2b}$$

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

where

L_{br} = unbraced length within the panel under consideration, in. (mm)

P_r = required axial strength of the column within the panel under consideration, using LRFD or ASD load combinations, kips (N)

2. Point Bracing

In the direction perpendicular to the longitudinal axis of the column, the required strength of end and intermediate point braces is

$$P_{br} = 0.01P_r \tag{A-6-3}$$

and, the required stiffness of the brace is

$$\beta_{br} = \frac{1}{\phi}\left(\frac{8P_r}{L_{br}}\right) \text{ (LRFD)} \tag{A-6-4a}$$

$$\beta_{br} = \Omega\left(\frac{8P_r}{L_{br}}\right) \text{ (ASD)} \tag{A-6-4b}$$

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

where

L_{br} = unbraced length adjacent to the point brace, in. (mm)

P_r = largest of the required axial strengths of the column within the unbraced lengths adjacent to the point brace using LRFD or ASD load combinations, kips (N)

When the unbraced lengths adjacent to a point brace have different P_r/L_{br} values, the larger value shall be used to determine the required brace stiffness.

For intermediate point bracing of an individual column, L_{br} in Equations A-6-4a or A-6-4b need not be taken less than the maximum effective length, L_c, permitted for the column based upon the required axial strength, P_r.

6.3. BEAM BRACING

Beams shall be restrained against rotation about their longitudinal axis at points of support. When a braced point is assumed in the design between points of support, lateral bracing, torsional bracing, or a combination of the two shall be provided to prevent the relative displacement of the top and bottom flanges (i.e., to prevent twist). In members subject to double curvature bending, the inflection point shall not be considered a braced point unless bracing is provided at that location.

The requirements of this section shall apply to bracing of doubly and singly symmetric I-shaped members subjected to flexure within a plane of symmetry and zero net axial force.

1. Lateral Bracing

Lateral bracing shall be attached at or near the beam compression flange, except as follows:

(a) At the free end of a cantilevered beam, lateral bracing shall be attached at or near the top (tension) flange.
(b) For braced beams subject to double curvature bending, bracing shall be attached at or near both flanges at the braced point nearest the inflection point.

It is permitted to use either panel or point bracing to provide lateral bracing for beams.

1a. Panel Bracing

The panel bracing system shall have the strength and stiffness specified in this section. The connection of the bracing system to the member shall have the strength specified in Section 6.3.1b for a point brace at that location.

User Note: The stiffness contribution of the connection to the panel bracing system should be assessed as provided in the User Note to Section 6.2.1.

The required shear strength of the bracing system is

$$V_{br} = 0.01\left(\frac{M_r C_d}{h_o}\right) \qquad \text{(A-6-5)}$$

and, the required shear stiffness of the bracing system is

$$\beta_{br} = \frac{1}{\phi}\left(\frac{4M_r C_d}{L_{br} h_o}\right) \text{ (LRFD)} \qquad \text{(A-6-6a)}$$

$$\beta_{br} = \Omega\left(\frac{4M_r C_d}{L_{br} h_o}\right) \text{ (ASD)} \qquad \text{(A-6-6b)}$$

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

where

C_d = 1.0, except in the following case:
= 2.0 for the brace closest to the inflection point in a beam subject to double curvature bending

L_{br} = unbraced length within the panel under consideration, in. (mm)

M_r = required flexural strength of the beam within the panel under consideration, using LRFD or ASD load combinations, kip-in. (N-mm)

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

1b. Point Bracing

In the direction perpendicular to the longitudinal axis of the beam, the required strength of end and intermediate point braces is

$$P_{br} = 0.02\left(\frac{M_r C_d}{h_o}\right) \quad \text{(A-6-7)}$$

and, the required stiffness of the brace is

$$\beta_{br} = \frac{1}{\phi}\left(\frac{10 M_r C_d}{L_{br} h_o}\right) \text{(LRFD)} \quad \text{(A-6-8a)}$$

$$\beta_{br} = \Omega\left(\frac{10 M_r C_d}{L_{br} h_o}\right) \text{(ASD)} \quad \text{(A-6-8b)}$$

ϕ = 0.75 (LRFD) Ω= 2.00 (ASD)

where

L_{br} = unbraced length adjacent to the point brace, in. (mm)

M_r = largest of the required flexural strengths of the beam within the unbraced lengths adjacent to the point brace using LRFD or ASD load combinations, kip-in. (N-mm)

When the unbraced lengths adjacent to a point brace have different M_r/L_{br} values, the larger value shall be used to determine the required brace stiffness.

For intermediate point bracing of an individual beam, L_{br} in Equations A-6-8a or A-6-8b need not be taken less than the maximum effective length, L_b, permitted for the beam based upon the required flexural strength, M_r.

2. Torsional Bracing

It is permitted to attach torsional bracing at any cross-section location, and it need not be attached near the compression flange.

User Note: Torsional bracing can be provided as point bracing, such as cross-frames, moment-connected beams or vertical diaphragm elements, or as continuous bracing, such as slabs or decks.

2a. Point Bracing

About the longitudinal axis of the beam, the required flexural strength of the brace is:

$$M_{br} = \frac{0.024 M_r L}{n C_b L_b} \quad \text{(A-6-9)}$$

and, the required flexural stiffness of the brace is:

$$\beta_{br} = \frac{\beta_T}{\left(1 - \frac{\beta_T}{\beta_{sec}}\right)} \quad \text{(A-6-10)}$$

where

$$\beta_T = \frac{1}{\phi}\frac{2.4L}{nEI_{yeff}}\left(\frac{M_r}{C_b}\right)^2 \quad \text{(LRFD)} \qquad \text{(A-6-11a)}$$

$$\beta_T = \Omega\frac{2.4L}{nEI_{yeff}}\left(\frac{M_r}{C_b}\right)^2 \quad \text{(ASD)} \qquad \text{(A-6-11b)}$$

$$\beta_{sec} = \frac{3.3E}{h_o}\left(\frac{1.5h_o t_w^3}{12} + \frac{t_{st} b_s^3}{12}\right) \qquad \text{(A-6-12)}$$

and

$\phi = 0.75$ (LRFD); $\Omega = 3.00$ (ASD)

User Note: $\Omega = 1.5^2/\phi = 3.00$ in Equations A-6-11a or A-6-11b, because the moment term is squared.

β_{sec} can be taken equal to infinity, and $\beta_{br} = \beta_T$, when a cross-frame is attached near both flanges or a vertical diaphragm element is used that is approximately the same depth as the beam being braced.

E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)
I_{yeff} = effective out-of-plane moment of inertia, in.^{4} (mm^4)
= $I_{yc} + (t/c)I_{yt}$
I_{yc} = moment of inertia of the compression flange about the y-axis, in.^{4} (mm^4)
I_{yt} = moment of inertia of the tension flange about the y-axis, in.^{4} (mm^4)
L = length of span, in. (mm)
M_r = largest of the required flexural strengths of the beam within the unbraced lengths adjacent to the point brace, using LRFD or ASD load combinations, kip-in. (N-mm)
$\frac{M_r}{C_b}$ = maximum value of the required flexural strength of the beam divided by the moment gradient factor, within the unbraced lengths adjacent to the point brace, using LRFD or ASD load combinations, kip-in. (N-mm)
b_s = stiffener width for one-sided stiffeners, in. (mm)
= twice the individual stiffener width for pairs of stiffeners, in. (mm)
c = distance from the neutral axis to the extreme compressive fibers, in. (mm)
n = number of braced points within the span
t = distance from the neutral axis to the extreme tensile fibers, in. (mm)
t_w = thickness of beam web, in. (mm)
t_{st} = thickness of web stiffener, in. (mm)
β_T = overall brace system required stiffness, kip-in./rad (N-mm/rad)
β_{sec} = web distortional stiffness, including the effect of web transverse stiffeners, if any, kip-in./rad (N-mm/rad)

User Note: 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.

User Note: For doubly symmetric members, $c = t$ and I_{yeff} = out-of-plane moment of inertia, I_y, in.^{4} (mm^4).

When required, a 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 is 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.

In Equation A-6-9, L_b need not be taken less than the maximum unbraced length permitted for the beam based upon the required flexural strength, M_r.

2b. Continuous Bracing

For continuous torsional bracing:

(a) The brace strength requirement per unit length along the beam shall be taken as Equation A-6-9 divided by the maximum unbraced length permitted for the beam based upon the required flexural strength, M_r. The required flexural strength, M_r, shall be taken as the maximum value throughout the beam span.

(b) The brace stiffness requirement per unit length shall be given by Equations A-6-10 and A-6-11 with $L/n = 1.0$.

(c) The web distortional stiffness shall be taken as:

$$\beta_{sec} = \frac{3.3Et_w^3}{12h_o} \qquad \text{(A-6-13)}$$

6.4. BEAM-COLUMN BRACING

For bracing of beam-columns, the required strength and stiffness for the axial force shall be determined as specified in Section 6.2, and the required strength and stiffness for flexure shall be determined as specified in Section 6.3. The values so determined shall be combined as follows:

(a) When panel bracing is used, the required strength shall be taken as the sum of the values determined using Equations A-6-1 and A-6-5, and the required stiffness shall be taken as the sum of the values determined using Equations A-6-2 and A-6-6.

(b) When point bracing is used, the required strength shall be taken as the sum of the values determined using Equations A-6-3 and A-6-7, and the required stiffness shall be taken as the sum of the values determined using Equations A-6-4 and A-6-8. In Equations A-6-4 and A-6-8, L_{br} for beam-columns shall be taken as the actual unbraced length; the provisions in Sections 6.2.2 and 6.3.1b, that L_{br} need not be taken less than the maximum permitted effective length based upon P_r and M_r, shall not be applied.

(c) When torsional bracing is provided for flexure in combination with panel or point bracing for the axial force, the required strength and stiffness shall be combined or distributed in a manner that is consistent with the resistance provided by the element(s) of the actual bracing details.

(d) When the combined stress effect from axial force and flexure results in compression to both flanges, either lateral bracing shall be added to both flanges or both flanges shall be laterally restrained by a combination of lateral and torsional bracing.

User Note: For case (d), additional guidelines are provided in the Commentary.

APPENDIX 7

ALTERNATIVE METHODS OF DESIGN FOR STABILITY

This appendix presents alternatives to the direct analysis method of design for stability defined in Chapter C. The two alternative methods covered are the effective length method and the first-order analysis method.

The appendix is organized as follows:

7.1. GENERAL STABILITY REQUIREMENTS

The general requirements of Section C1 shall apply. As an alternative to the direct analysis method (defined in Sections C1 and C2), it is permissible to design structures for stability in accordance with either the effective length method, specified in Section 7.2, or the first-order analysis method, specified in Section 7.3, subject to the limitations indicated in those sections.

7.2. EFFECTIVE LENGTH METHOD

1. Limitations

When using the effective length method, the following conditions shall be met:

(a) The structure supports gravity loads primarily through nominally vertical columns, walls or frames.

(b) The ratio of maximum second-order drift to maximum first-order drift (both determined for load and resistance factor design (LRFD) load combinations or 1.6 times allowable strength design (ASD) load combinations, with stiffness not adjusted as specified in Section C2.3) in all stories is equal to or less than 1.5.

User Note: The ratio of second-order drift to first-order drift in a story may be taken as the B_2 multiplier, calculated as specified in Appendix 8.

2. Required Strengths

The required strengths of components shall be determined from an elastic analysis conforming to the requirements of Section C2.1, except that the stiffness reduction indicated in Section C2.1(a) shall not be applied; the nominal stiffnesses of all structural steel components shall be used. Notional loads shall be applied in the analysis in accordance with Section C2.2b.

User Note: Since the condition specified in Section C2.2b(d) will be satisfied in all cases where the effective length method is applicable, the notional load need only be applied in gravity-only load cases.

3. Available Strengths

The available strengths of members and connections shall be calculated in accordance with the provisions of Chapters D through K, as applicable.

For flexural buckling, the effective length, L_c, of members subject to compression shall be taken as KL, where K is as specified in (a) or (b), in the following, as applicable, and L is the laterally unbraced length of the member.

(a) In braced-frame systems, shear-wall systems, and other structural systems where lateral stability and resistance to lateral loads does not rely on the flexural stiffness of columns, the effective length factor, K, of members subject to compression shall be taken as unity unless a smaller value is justified by rational analysis.

(b) In moment-frame systems and other structural systems in which the flexural stiffnesses of columns are considered to contribute to lateral stability and resistance to lateral loads, the effective length factor, K, or elastic critical buckling stress, F_e, of those columns whose flexural stiffnesses are considered to contribute to lateral stability and resistance to lateral loads shall be determined from a sidesway buckling analysis of the structure; K shall be taken as 1.0 for columns whose flexural stiffnesses are not considered to contribute to lateral stability and resistance to lateral loads.

Exception: It is permitted to use $K = 1.0$ in the design of all columns if the ratio of maximum second-order drift to maximum first-order drift (both determined for LRFD load combinations or 1.6 times ASD load combinations) in all stories is equal to or less than 1.1.

User Note: Methods of calculating the effective length factor, K, are discussed in the Commentary.

Bracing intended to define the unbraced lengths of members shall have sufficient stiffness and strength to control member movement at the braced points.

User Note: Methods of satisfying the bracing requirement are provided in Appendix 6. The requirements of Appendix 6 are not applicable to bracing that is included in the design of the lateral force-resisting system of the overall structure.

7.3. FIRST-ORDER ANALYSIS METHOD

1. Limitations

When using the first-order analysis method, the following conditions shall be met:

(a) The structure supports gravity loads primarily through nominally vertical columns, walls or frames.

(b) The axial forces in nominally horizontal members in moment frames are not larger than $0.1F_eA_g$ with L_c taken as the unbraced length of the member.

(c) The ratio of maximum second-order drift to maximum first-order drift (both determined for LRFD load combinations or 1.6 times ASD load combinations, with stiffness not adjusted as specified in Section C2.3) in all stories is equal to or less than 1.5.

User Note: The ratio of second-order drift to first-order drift in a story may be taken as the B_2 multiplier, calculated as specified in Appendix 8.

(d) The required axial compressive strengths of all members whose flexural stiffnesses are considered to contribute to the lateral stability of the structure satisfy the limitation:

$$\alpha P_r \le 0.5P_{ns} \quad \text{(A-7-1)}$$

where

α = 1.0 (LRFD); α = 1.6 (ASD)
P_r = required axial compressive strength under LRFD or ASD load combinations, kips (N)
P_{ns} = cross-section compressive strength; for nonslender-element sections, $P_{ns} = F_y A_g$, and for slender-element sections, $P_{ns} = F_y A_e$, where A_e is as defined in Section E7, kips (N)

2. Required Strengths

The required strengths of components shall be determined from a first-order analysis, with additional requirements (a) and (b) given in the following. The analysis shall consider flexural, shear and axial member deformations, and all other deformations that contribute to displacements of the structure.

(a) All load combinations shall include an additional lateral load, N_i, applied in combination with other loads at each level of the structure:

$$N_i = 2.1\alpha(\Delta/L)Y_i \ge 0.0042Y_i \quad \text{(A-7-2)}$$

where

α = 1.0 (LRFD); α = 1.6 (ASD)
Y_i = gravity load applied at level i from the LRFD load combination or ASD load combination, as applicable, kips (N)
Δ/L = maximum ratio of Δ to L for all stories in the structure
Δ = first-order interstory drift due to the LRFD or ASD load combination, as applicable, in. (mm). Where Δ varies over the plan area of the structure, Δ shall be the average drift weighted in proportion to vertical load or, alternatively, the maximum drift.
L = height of story, in. (mm)

The additional lateral load at any level, N_i, shall be distributed over that level in the same manner as the gravity load at the level. The additional lateral loads shall be applied in the direction that provides the greatest destabilizing effect.

> **User Note:** For most building structures, the requirement regarding the direction of N_i may be satisfied as follows: (a) For load combinations that do not include lateral loading, consider two alternative orthogonal directions for the additional lateral load in a positive and a negative sense in each of the two directions, same direction at all levels; (b) for load combinations that include lateral loading, apply all the additional lateral loads in the direction of the resultant of all lateral loads in the combination.

(b) The nonsway amplification of beam-column moments shall be included by applying the B_1 amplifier of Appendix 8 to the total member moments.

> **User Note:** Since there is no second-order analysis involved in the first-order analysis method for design by ASD, it is not necessary to amplify ASD load combinations by 1.6 before performing the analysis, as required in the direct analysis method and the effective length method.

3. Available Strengths

The available strengths of members and connections shall be calculated in accordance with the provisions of Chapters D through K, as applicable.

The effective length for flexural buckling of all members shall be taken as the unbraced length unless a smaller value is justified by rational analysis.

Bracing intended to define the unbraced lengths of members shall have sufficient stiffness and strength to control member movement at the braced points.

> **User Note:** Methods of satisfying this requirement are provided in Appendix 6. The requirements of Appendix 6 are not applicable to bracing that is included in the analysis of the overall structure as part of the overall force-resisting system.

APPENDIX 8

APPROXIMATE ANALYSIS

This appendix provides approximate analysis procedures for determining the required strength of structural members and connections.
The appendix is organized as follows:

8.1. Approximate Second-Order Elastic Analysis
8.2. Approximate Inelastic Moment Redistribution

8.1. APPROXIMATE SECOND-ORDER ELASTIC ANALYIS

Second-order effects in structures may be approximated by amplifying the required strengths determined by two first-order elastic analyses. The use of this procedure is limited to structures that support gravity loads primarily through nominally vertical columns, walls or frames, except that it is permissible to use the procedure specified for determining P-δ effects for any individual compression member. This method is not permitted for design by advanced analysis using the provisions of Appendix 1.

1. Calculation Procedure

The required second-order flexural strength, M_r, and axial strength, P_r, of all members shall be determined as:

$$M_r = B_1 M_{nt} + B_2 M_{lt} \quad \text{(A-8-1)}$$

$$P_r = P_{nt} + B_2 P_{lt} \quad \text{(A-8-2)}$$

where

B_1 = multiplier to account for P-δ effects, determined for each member subject to compression and flexure, and each direction of bending of the member in accordance with Appendix 8, Section 8.1.2. B_1 shall be taken as 1.0 for members not subject to compression.

B_2 = multiplier to account for P-Δ effects, determined for each story of the structure and each direction of lateral translation of the story in accordance with Appendix 8, Section 8.1.3.

M_{lt} = first-order moment using LRFD or ASD load combinations, due to lateral translation of the structure only, kip-in. (N-mm)

M_{nt} = first-order moment using LRFD or ASD load combinations, with the structure restrained against lateral translation, kip-in. (N-mm)

M_r = required second-order flexural strength using LRFD or ASD load combinations, kip-in. (N-mm)

P_{lt} = first-order axial force using LRFD or ASD load combinations, due to lateral translation of the structure only, kips (N)

P_{nt} = first-order axial force using LRFD or ASD load combinations, with the structure restrained against lateral translation, kips (N)

P_r = required second-order axial strength using LRFD or ASD load combinations, kips (N)

User Note: Equations A-8-1 and A-8-2 are applicable to all members in all structures. Note, however, that B_1 values other than unity apply only to moments in beam-columns; B_2 applies to moments and axial forces in components of the lateral force-resisting system (including columns, beams, bracing members and shear walls). See the Commentary for more on the application of Equations A-8-1 and A-8-2.

2. Multiplier B_1 for P-δ Effects

The B_1 multiplier for each member subject to compression and each direction of bending of the member is calculated as:

$$B_1 = \frac{C_m}{1 - \alpha P_r / P_{e1}} \geq 1 \tag{A-8-3}$$

where

α = 1.0 (LRFD); α = 1.6 (ASD)

C_m = equivalent uniform moment factor, assuming no relative translation of the member ends, determined as follows:

(a) For beam-columns not subject to transverse loading between supports in the plane of bending

$$C_m = 0.6 - 0.4\left(M_1/M_2\right) \tag{A-8-4}$$

where M_1 and M_2, calculated from a first-order analysis, are the smaller and larger moments, respectively, at the ends of that portion of the member unbraced in the plane of bending under consideration. M_1/M_2 is positive when the member is bent in reverse curvature, and negative when bent in single curvature.

(b) For beam-columns subject to transverse loading between supports, the value of C_m shall be determined either by analysis or conservatively taken as 1.0 for all cases.

P_{e1} = elastic critical buckling strength of the member in the plane of bending, calculated based on the assumption of no lateral translation at the member ends, kips (N)

$$= \frac{\pi^2 EI^*}{\left(L_{c1}\right)^2} \tag{A-8-5}$$

where

EI^* = flexural rigidity required to be used in the analysis (= $0.8\tau_b EI$ when used in the direct analysis method, where τ_b is as defined in Chapter C; = EI for the effective length and first-order analysis methods)

E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)

I = moment of inertia in the plane of bending, in.^{4} (mm^4)

L_{c1} = effective length in the plane of bending, calculated based on the assumption of no lateral translation at the member ends, set equal to the laterally unbraced length of the member unless analysis justifies a smaller value, in. (mm)

It is permitted to use the first-order estimate of P_r (i.e., $P_r = P_{nt} + P_{lt}$) in Equation A-8-3.

3. Multiplier B_2 for P-Δ Effects

The B_2 multiplier for each story and each direction of lateral translation is calculated as:

$$B_2 = \frac{1}{1 - \dfrac{\alpha P_{story}}{P_{e\,story}}} \geq 1 \qquad \text{(A-8-6)}$$

where

α = 1.0 (LRFD); α = 1.6 (ASD)

P_{story} = total vertical load supported by the story using LRFD or ASD load combinations, as applicable, including loads in columns that are not part of the lateral force-resisting system, kips (N)

$P_{e\,story}$ = elastic critical buckling strength for the story in the direction of translation being considered, kips (N), determined by side-sway buckling analysis or as:

$$= R_M \frac{H\,L}{\Delta_H} \qquad \text{(A-8-7)}$$

and

H = total story shear, in the direction of translation being considered, produced by the lateral forces used to compute Δ_H, kips (N)

L = height of story, in. (mm)

R_M = $1 - 0.15\,(P_{mf}/P_{story})$ (A-8-8)

P_{mf} = total vertical load in columns in the story that are part of moment frames, if any, in the direction of translation being considered (= 0 for braced-frame systems), kips (N)

Δ_H = first-order interstory drift, in the direction of translation being considered, due to lateral forces, in. (mm), computed using the stiffness required to be used in the analysis. (When the direct analysis method is used, stiffness is reduced according to Section C2.3.) Where Δ_H varies over the plan area of the structure, it shall be the average drift weighted in proportion to vertical load or, alternatively, the maximum drift.

User Note: R_M can be taken as 0.85 as a lower bound value for stories that include moment frames, and R_M = 1 if there are no moment frames in the story. H and Δ_H in Equation A-8-7 may be based on any lateral loading that provides a representative value of story lateral stiffness, H/Δ_H.

8.2. APPROXIMATE INELASTIC MOMENT REDISTRIBUTION

The required flexural strength of indeterminate beams comprised of compact sections, as defined in Section B4.1, carrying gravity loads only, and satisfying the unbraced length requirements provided below, is permitted to be taken as nine-tenths of the negative moments at the points of support, produced by the gravity loading and determined by an elastic analysis satisfying the requirements of Chapter C, provided that the maximum positive moment is increased by one-tenth of the average negative moment determined by an elastic analysis. This moment redistribution is not permitted for moments in members with F_y exceeding 65 ksi (450 MPa), for moments produced by loading on cantilevers, for design using partially restrained (PR) moment connections, or for design by inelastic analysis using the provisions of Appendix 1.2. This moment redistribution is permitted for design according to Section B3.1 (LRFD) and for design according to Section B3.2 (ASD). The required axial strength shall not exceed $0.15\phi_c F_y A_g$ for LRFD or $0.15F_yA_g/\Omega_c$ for ASD, where ϕ_c and Ω_c are determined from Section E1, A_g = gross area of member, in.^{2} (mm^2), and F_y = specified minimum yield stress, ksi (MPa).

The laterally unbraced length, L_b, of the compression flange adjacent to the redistributed end moment locations shall not exceed L_m determined as follows.

(a) For doubly symmetric and singly symmetric I-shaped beams with the I_{yc} of the compression flange equal to or larger than the I_{yt} of the tension flange loaded in the plane of the web

$$L_m = \left[0.12 + 0.076\left(\frac{M_1}{M_2}\right)\right]\left(\frac{E}{F_y}\right)r_y \qquad \text{(A-8-9)}$$

(b) For solid rectangular bars and symmetric box beams bent about their major axis

$$L_m = \left[0.17 + 0.10\left(\frac{M_1}{M_2}\right)\right]\left(\frac{E}{F_y}\right)r_y \geq 0.10\left(\frac{E}{F_y}\right)r_y \qquad \text{(A-8-10)}$$

where

F_y = specified minimum yield stress of the compression flange, ksi (MPa)
M_1 = smaller moment at end of unbraced length, kip-in. (N-mm)
M_2 = larger moment at end of unbraced length, kip-in. (N-mm)
r_y = radius of gyration about y-axis, in. (mm)
(M_1/M_2) is positive when moments cause reverse curvature and negative for single curvature

There is no limit on L_b for members with round or square cross sections or for any beam bent about its minor axis.

APPENDIX X

DESIGN OF FILLED COMPOSITE MEMBERS WITH HIGH-STRENGTH MATERIALS

Comment [HC1]: New Appendix is proposed.

This appendix provides methods for calculating the design strength of filled composite members with high-strength materials. These provisions shall be used in lieu of Sections I1.2, I1.3, I1.4, I2, I3, and I5. Other provisions of Chapter I shall apply.

X.1. RECTANGULAR FILLED COMPOSITE MEMBERS

1. Limitations

For rectangular filled composite members, the following limitations shall be met:

(a) The area of the steel section shall comprise at least 1% of the total composite cross section.

(b) Concrete shall be normal weight, and compressive strength, $f_c^{'}$, shall not exceed 15 ksi (103 MPa).

(c) The specified minimum yield stress of steel shall not exceed 100 ksi (690 MPa).

(d) The maximum permitted width-to-thickness ratio for compression steel elements shall be limited to $5.00\sqrt{E/F_y}$.

(e) Longitudinal reinforcement is not required. If longitudinal refinforcement is provided, it shall not be considered in the calculation of available strength.

2. Compressive Strength

The available compressive strength shall be determined in accordance with Section I2.2b with the following modifications:

$$P_{no} = F_{cr} A_s + 0.85 f_c^{'} A_c \qquad \text{(A-X-1)}$$

$$F_{cr} = 1.0 - 0.075 \lambda F_y \qquad \text{(A-X-2)}$$

where

λ = largest width-to-thickness ratio of compression steel elements

3. Flexural Strength

The available flexural strength shall be determined in as follows:

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

The nominal flexural strength, M_n, shall be determined as 90% of the moment corresponding to a plastic stress distribution over the composite cross-section assuming that steel components have reached a stress of either F_y in tension or F_{cr} in compression, where F_{cr} calculated using Equation A-X-2, and concrete components in compression have reached a stress of $0.85 f_c'$, where f_c' is the specified compressive strength of concrete, ksi (MPa).

4. Combined Flexure and Axial Force

The interaction of flexure and compression shall be limited by Equations I5-1a and I5-1b where the term c_p is determined using Equation A-X-3 and c_m is determined using A-X-4.

$$c_p = 0.175 - \frac{0.075}{B/H} + \lambda\left(\frac{0.3}{P_n/P_{no}}\right)\left(\frac{f_c'}{F_y}\right) \qquad \text{(A-X-3)}$$

$$c_m = 0.6 + 0.3\left(\frac{P_n}{P_{no}}\right)^2 + 0.6\lambda\left(\frac{B}{H}\right)\left(\frac{F_y^{\max}}{F_y}\right)\left(\frac{f_c'}{F_y}\right) \qquad \text{(A-X-4)}$$

where

B = flange width of rectangular cross section
H = web depth of rectangular cross section
F_y^{max} = maximum permitted yield stress of steel = 100 ksi (690 MPa)
P_n = nominal axial strength calculated in accordance with Section X.1.1
t = flange thickness