ANSI/AISC 370-21
An American National Standard

Specification for Structural Stainless Steel Buildings

June 11, 2021
Approved by the Committee on Structural Stainless Steel

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Stronger.
Steel.

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PREFACE

(This Preface is not part of ANSI/AISC 370-21, *Specification for Structural Stainless Steel Buildings*, but is included for informational purposes only.)

This is the first edition of the *Specification for Structural Stainless Steel Buildings*. Similar to the AISC *Specification for Structural Steel Buildings*, ANSI/AISC 360-16, this Specification provides an integrated treatment of allowable strength design (ASD) and load and resistance factor design (LRFD). As indicated in Chapter B of the Specification, designs can be made according to either ASD or LRFD provisions.

There are a wide array of stainless steel types available for structural applications. The properties of stainless steels differ significantly from the properties of the steels covered in the AISC *Specification for Structural Steel Buildings*. That specification is not appropriate for the design and fabrication of structural stainless steel. Prior to this Specification there was no specification for the use of stainless steel in building structural applications. The first edition of AISC Design 27, *Structural Stainless Steel*, provided some guidance on the design and fabrication of structural stainless steel buildings; that design guide is being updated, and the second edition will serve as a companion to AISC 313-21 and ANSI/AISC 370-21.

This ANSI-approved Specification has been developed as a consensus document using ANSI-accredited procedures to provide a uniform practice in the design of structural stainless steel-framed buildings and other structures. The intention is to provide design criteria for routine use and not to provide specific criteria for infrequently encountered problems, which occur in the full range of structural design.

This Specification is the result of the consensus deliberations of a committee of structural engineers with approximately equal numbers in private practice, in research and testing, and employed by stainless steel fabricating and producing companies.

The Symbols, Glossary, Abbreviations, and Appendices to this Specification are an integral part of the Specification. A nonmandatory Commentary has been prepared to provide background for the Specification provisions, and the user is encouraged to consult it. Additionally, nonmandatory User Notes are interspersed throughout the Specification to provide concise and practical guidance in the application of the provisions.

This Specification was approved by the AISC Committee on Structural Stainless Steel,

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The Committee also gratefully acknowledges AISC Board Oversight, Matt Smith; the advisory members, Javier Avila Mendoza, Nancy Baddoo, Leroy Gardner, and Ben Young; as well as Francisco Meza, for their contributions and involvement in the development of this document.

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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 righthand 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}	Cross-sectional 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_{corner}	Total corner area including a region of length $2t$ extending around the perimeter of the cross section on both sides of the each corner, in.^{2} (mm^2)	B4.3
A_e	Effective area, in.^{2} (mm^2)	E7.2
A_e	Effective net area, in.^{2} (mm^2)	D2
A_e	Summation of the effective areas of the cross section based on the reduced effective widths, b_e, d_e, or h_e, in.^{2} (mm^2)	E7
A_{fc}	Area of compression flange, in.^{2} (mm^2)	G2.2
A_{fg}	Gross area of tension flange, in.^{2} (mm^2)	F11.1
A_{fn}	Net area of tension flange, in.^{2} (mm^2)	F11.1
A_{ft}	Area of tension flange, in.^{2} (mm^2)	G2.2
A_g	Gross area of member, in.^{2} (mm^2)	B4.3
A_{gv}	Gross area subject to shear, in.^{2} (mm^2)	J4.2
A_n	Net area of member, in.^{2} (mm^2)	B4.4b
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_p	Gross area of pin, in.^{2} (mm^2)	J8.2
A_{pb}	Projected area in bearing, in.^{2} (mm^2)	J7
A_s	Net tensile area of bolt, in.^{2} (mm^2)	J3.8
A_{sf}	Area on the shear failure path, in.^{2} (mm^2)	D5.1
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_w	Area of web, in.^{2} (mm^2)	G2.1
A_{we}	Effective area of the weld, in.^{2} (mm^2)	J2.4
A_1	Area of steel concentrically bearing on a concrete support, in.^{2} (mm^2)	J9
A_2	Maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, in.^{2} (mm^2)	J9
B	Overall width of square and rectangular HSS, in. (mm)	B4.3
B	Overall width of rectangular HSS main member, measured 90° to the plane of the connection, in. (mm)	Table D3.1
B	Overall width of square HSS main member, measured 90° to plane of the connection, in. (mm)	K2.1
B	Overall width of HSS and box-section members, in. (mm)	App. 2.7.1

Symbol	Definition	Section
B_b	Overall width of square HSS branch member or plate, measured 90° to the plane of the connection, in. (mm)	K2.1
B_e	Effective width of square HSS branch member or plate, in. (mm)	K2.1
C	HSS torsional constant	G8
C	Coefficient from Table J12.1	J12
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_h	Web slenderness coefficient from Table J12.1	J12
C_l	Bearing length coefficient from Table J12.1	J12
C_r	Internal bend radius coefficient from Table J12.1	J12
C_v	Shear buckling strength coefficient	G4
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_2	Edge distance increment, in. (mm)	Table J3.3
D	Diameter of round HSS, in. (mm)	B4.1b
D	Outside diameter of round HSS, in. (mm)	B4.3
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)	K2.1
D_{LT}	Lateral-torsional buckling coefficient	F2.2
D_u	In slip-critical connections, 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 stainless steel, ksi (MPa)	Table B4.1
E_{LT}	Lateral-torsional buckling coefficient	F4.2
E_r	Reduced modulus of elasticity, ksi (MPa)	L3
E_s	Secant modulus, ksi (MPa)	L3
E_{sc}	Secant modulus corresponding to the maximum compressive stress in the cross section, ksi (MPa)	L3
E_{sh}	Strain hardening modulus, ksi (MPa)	App. 2.2
E_{st}	Secant modulus corresponding to the maximum tensile stress in the cross section, ksi (MPa)	L3
E_t	Tangent modulus, ksi (MPa)	App. 7.1.2
E_{ty}	Tangent modulus at the specified minimum yield stress, ksi (MPa)	App. 7.1.1
$E(T)$	Modulus of elasticity at elevated temperature, ksi (MPa)	App. 4.2.4d
$E(T)$	Modulus of elasticity of stainless steel at elevated temperatures, ksi (MPa)	App. 7.2
$E_{Ty}(T)$	Tangent modulus at the yield stress at elevated temperatures, ksi (MPa)	App 7.2
F_c	Available stress in main member, ksi (MPa)	K2.1
F_{cr}	Buckling stress for the section as determined by analysis, ksi (MPa)	G9
F_{cr}	Critical stress, ksi (MPa)	E3
F_{cr}	Lateral-torsional buckling stress for the section as determined by analysis, ksi (MPa)	F10.2

Symbol	Definition	Section
F_{cr}	Local buckling stress for the section as determined by analysis, ksi (MPa)	F10.3
F_e	Elastic buckling stress, ksi (MPa)	E3
F_{el}	Elastic local buckling stress, ksi (MPa)	E7.1
F_{EXX}	Filler metal classification strength, ksi (MPa)	J2.4
F_{LT}	Lateral-torsional buckling coefficient.	F5.2
F_n	Nominal tensile stress or shear stress, ksi (MPa)	J2.6
F_{nBM}	Nominal stress of the base metal, ksi (MPa)	J2.4
F_{nt}	Nominal tensile stress, ksi (MPa)	J3.6
F'_{nt}	Nominal tensile stress modified to include the effects of shear stress, ksi (MPa)	J3.7
F_{nv}	Nominal shear stress, ksi (MPa)	J3.6
F_{nw}	Nominal stress of the weld metal, ksi (MPa)	J2.4
F_{ser}	Maximum serviceability design stress, ksi (MPa)	L3
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)	B4.3
F_y	Specified minimum yield stress, ksi (MPa). As used in this Specification, “yield stress” denotes specified yield strength	Table B4.1
$F_{y,avg}$	Average yield stress of full section, ksi (MPa)	B4.3
F_{yb}	Specified minimum yield strength of bolt, ksi (MPa)	J3.8
F_{yb}	Specified minimum yield stress of HSS branch member or plate material, ksi (MPa)	K2.1
$F_{y,corner}$	Tensile yield stress of corners of rectangular HSS, ksi (MPa)	B4.3
F_{yf}	Specified minimum yield stress of the flange, ksi (MPa)	J11.1
$F_{y,rnd}$	Tensile yield stress of round HSS, ksi (MPa)	B4.3
F_{yst}	Specified minimum yield stress of the stiffener material, ksi (MPa).	G2.3
F_{yw}	Specified minimum yield stress of the web material, ksi (MPa)	G2.3
$F_{y,wall}$	Tensile yield stress of the flats of rectangular HSS, ksi (MPa)	B4.3
$F_e(T)$	Critical elastic buckling stress, ksi (MPa)	App. 4.2.4d
$F_u(T)$	Tensile strength at elevated temperatures, ksi (MPa)	App. 4.2.4d
$F_y(T)$	Yield stress at elevated temperatures, ksi (MPa)	App. 4.2.4d
$F_2(T)$	Stress at 2% strain at elevated temperatures, ksi (MPa)	App. 4.2.3b
G	Shear modulus of elasticity of stainless steel, ksi (MPa)	E4
H	Flexural constant.	E4
H	Overall height of square and rectangular HSS, in. (mm)	B4.3
H	Overall height of rectangular HSS member, measured in the plane of the connection, in. (mm)	Table D3.1
H	Overall height of HSS and box-section member, in. (mm)	App. 2.7.1
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)	G2.3
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)	G2.3

Symbol	Definition	Section
I_{st2}	Minimum moment of inertia of transverse stiffeners required for development of web shear buckling resistance, in.^{4} (mm^4)	G2.3
I_x, I_y	Moment of inertia about the principal axes, in.^{4} (mm ^{4})	E4
I_y	Moment of inertia about 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)	G8.1
L	Laterally unbraced length of member, in. (mm)	E2
L	Laterally unbraced length, in. (mm)	J4.4
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_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	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)	F9.2
L_b	Largest laterally unbraced length along either flange at the point of load, in. (mm)	J11.4
L_b	Laterally unbraced length, in. (mm)	App. 1.3.2b
L_{br}	Unbraced length within the panel under consideration, in. (mm)	App. 6.3.1a
L_{br}	Unbraced length adjacent to the point brace, in. (mm).	App. 6.3.1b
L_c	Effective length of member, in. (mm)	E2
L_c	Effective length of member for buckling about minor axis, in. (mm).	E5
L_c	Effective length, in. (mm)	J4.4
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_{el}	Local buckling half-wavelength (mm).	App. 1.3.3
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

Symbol	Definition	Section
L_r	Nominal roof live load	App. 5.4.1
L_v	Distance from maximum to zero shear force, in. (mm)	G4
L_x, L_y, L_z	Laterally unbraced length of the member for each axis, in. (mm)	E4
L_y	Laterally unbraced length required to achieve the yield moment, in. (mm)	F2.2
M_A	Absolute value of moment at one-quarter point of the unbraced segment, kip-in. (N-mm)	F1
M_a	Required flexural strength using ASD load combinations, kip-in. (N-mm)	J11.4
M_B	Absolute value of moment at centerline of the unbraced segment, kip-in. (N-mm)	F1
M_{br}	Required flexural strength of the brace, kip-in. (N-mm)	App. 6.3.2a
M_C	Absolute value of moment at three-quarter point of the unbraced segment, kip-in. (N-mm)	F1
M_c	Available flexural strength, kip-in. (N-mm)	H1.1
M_c	Available CSM flexural strength, kip-in. (N-mm)	App. 2.7
$M_{c,red}$	Available reduced CSM flexural strength, kip-in. (N-mm)	App. 2.7.1
M_{cx}	Available flexural strength about *x*-axis for the limit state of tensile rupture of the flange, determined according to Section F10.1, kip-in. (N-mm)	H3
M_{max}	Absolute value of maximum moment in the unbraced segment, kip-in. (N-mm)	F1
M_{mid}	Moment at middle of unbraced length, kip-in. (N-mm)	App. 1.3.2b
M_n	Nominal flexural strength, kip-in. (N-mm)	F1
M_n	Nominal CSM flexural strength at the limit state of yielding or local buckling, kip-in. (N-mm)	App. 2.6
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 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, kip-in. (N-mm)	H3
M_u	Required flexural strength using LRFD load combinations, kip-in. (N-mm)	J11.4
M_y	Yield moment, kip-in. (N-mm)	F2.2
M_1	Smaller moment at end of unbraced length, kip-in. (N-mm)	App. 1.3.2b
M_1'	Effective moment at the end of the unbraced length opposite from M_2, kip-in. (N-mm)	App. 1.3.2b
M_2	Larger moment at end of unbraced length, kip-in. (N-mm)	App. 1.3.2b
N_i	Notional load applied at level *i*, kips (N)	C2.2b

Symbol	Definition	Section
P_{br}	Largest of the required axial strength 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_c	Available compressive strength determined in accordance with Chapter E, kips (N)	H1.1
P_c	Available tensile strength determined in accordance with Chapter D, kips (N)	H1.2
P_c	Available tensile or compressive strength determined in accordance with Chapter D or E, kips (N)	H2.1
P_c	Available axial strength for the limit state of tensile rupture of the net section at the location of bolt holes, determined in accordance with Section D2(b), kips (N)	H3
P_c	Available CSM compressive strength, kips (N)	App. 2.7
P_n	Nominal compressive strength, kips (N)	E1
P_{ns}	Cross-section compressive strength, kips (N)	C2.3
P_p	Nominal bearing strength, kips (N)	J9
P_r	Required axial compressive strength using LRFD or ASD load combinations, kips (N)	C2.3
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 combination, kips (N)	H2.2
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, kips (N)	H3
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	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_y	Axial yield strength of the column, kips (N)	J11.6
R	Radius of outside surface, in. (mm)	Table J2.2
R	Nominal load due to rainwater or snow, exclusive of the ponding contribution	App. 5.4.1
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_n	Nominal strength	B3.1
R_n	Combined strength of fillet weld group, kips (N)	J2.4
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

Symbol	Definition	Section
R_p	Web plastification factor	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_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	Elastic section modulus of pin, in.^{3} (mm^3)	J8.3
S	Nominal snow load, kips (N)	App. 4.1.4
S_e	Effective section modulus with respect to the neutral axis of the effective cross section, in.^{3} (mm^3)	F7.2
S_{min}	Minimum elastic section modulus relative to the axis of bending, in.^{3} (mm^3)	F10
S_x	Minimum elastic section modulus taken about the x-axis, in.^{3} (mm^3)	F11.1
S_x	Elastic section modulus taken about the x-axis, in.^{3} (mm^3)	F2.2
S_y	Elastic section modulus taken about the y-axis, in.^{3} (mm^3)	F6.1
S_{ye}	Effective section modulus taken about the y-axis with respect to the neutral axis of the effective cross section, in.^{3} (mm^3)	F6.2
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 (N)	J3.9
T_b	Minimum fastener tension for stainless steel bolts, kips (N)	J3.8
T_c	Available torsional strength determined in accordance with Section H2.1, kip-in. (N-mm)	H2.2
T_n	Nominal torsional strength, kip-in. (N-mm)	G1
T_r	Required torsional strength, determined in accordance with Chapter C, using LRFD or ASD load combinations, kip-in. (N-mm)	H2.2
T_u	Required tension force using LRFD load combinations, kips (N)	J3.9
U	Shear lag factor	D3
V_{br}	Required shear strength of the bracing system, kips (N)	App. 6.2.1
V_c	Available shear strength, kips (N)	H2.2
V_{c1}	Available shear strength, kips (N)	G2.3
V_{c2}	Available shear strength, kips (N)	G2.3
V_n	Nominal shear strength, kips (N)	G1
V_r	Required shear strength in the panel being considered, kips (N)	G2.3
V_r	Required shear strength, determined in accordance with Chapter C, using LRFD or ASD load combinations, kips (N)	H2.2
Y_i	Gravity load applied at level i from the LRFD load combination or ASD load combination, as applicable, kips (N)	C2.2b
Y_i	Gravity load acting on framing level i, kips (N)	App. 4.1.4
Z	Plastic section modulus taken about the axis of bending, in.^{3} (mm^3)	F7.1
Z_x	Plastic section modulus taken about the x-axis, in.^{3} (mm^3)	F2.1
Z_y	Plastic section modulus taken about the y-axis, in.^{3} (mm^3)	F6.1
a	Clear distance between transverse stiffeners, in. (mm)	G2.1
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

Symbol	Definition	Section
a_f	Ratio of the flange area to the gross area of member	App. 2.7.1
a_w	Ratio of the web area to the gross area of member	App. 2.7.1
b	Width of element, in. (mm)	B4.1a
b	Distance from face of plate to edge of rectangular HSS member, measured 90° to the plane of the connection, in. (mm)	Table D3.1
b	Width of compression flange, in. (mm)	F7.2
b_{cf}	Width of column flange, in. (mm)	J11.6
b_e	Effective width, in. (mm)	E7.1
b_e	Distance from edge of hole to edge of part, measured in direction normal to applied force, in. (mm)	D5.1
b_f	Width of flange, in. (mm)	B4.1a
b_{fc}	Width of compression flange, in. (mm)	G2.2
b_{ft}	Width of tension flange, in. (mm)	G2.2
b_p	Smaller of the dimension a and h, in. (mm)	G2.3
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
d	For stems of tees, the full depth of section, in. (mm)	B4.1a
d	Depth of section, in. (mm)	Table D3.1
d	Depth of section from which the tee was cut, in. (mm)	Table D3.1
d	Nominal diameter of fastener, in. (mm)	J3.3
d	Full nominal depth of member, in. (mm)	J11.2
d_b	Depth of beam, in. (mm)	J11.6
d_b	Nominal diameter (body or shank diameter), in. (mm)	App. 3.4
d_c	Depth of column, in. (mm)	J11.6
d_e	Effective width for tees, in. (mm)	E7.1
d_h	Diameter of hole, in. (mm)	J3.10
d_{pin}	Diameter of pin, in. (mm)	D5.1
e	Eccentricity in a truss connection, positive being away from the branches, in. (mm)	K2.1
f	Stress at outer compressive fiber, ksi (MPa)	App. 1.3.3d
f	Engineering stress, ksi (MPa)	App. 7.1.1
f'_c	Specified compressive strength of concrete, ksi (MPa)	J9
f_{csm}	Stress corresponding to ε_{csm}, ksi (MPa)	App. 2.5
$f_{csm,t}$	Tensile stress corresponding to $15\varepsilon_y$, ksi (MPa)	App. 2.4
f_{rv}	Required shear stress using LRFD or ASD load combinations, ksi (MPa)	J3.7
$f(T)$	Engineering stress at elevated temperatures, ksi (MPa)	App. 7.2
g	Transverse center-to-center spacing (gage) between fastener gage lines, in. (mm)	B4.4b
g	Gap between toes of branch members in a gapped K-connection, neglecting the welds, in. (mm)	K2.1
h	Depth of web, in. (mm)	B4.1b
h	Width resisting the shear force, in. (mm)	G3
h	Flat width of longer side, in. (mm)	G8.2

Symbol	Definition	Section
h	For rolled I-shaped members, the clear distance between flanges less the fillet at each flange; for built-up welded sections or members, the clear distance between flanges; for built-up bolted members, the distance between fastener lines, in. (mm)	G2.1
h	Clear distance between flanges less corner radius, in. (mm)	J12
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 or members, in. (mm)	B4.1b
h_e	Effective width for webs, in. (mm)	E7.1
h_f	Factor for fillers	J3.8
h_o	Distance between flange centroids, in. (mm)	F2.2
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 for rolled sections or the thickness of flange for welded sections, in. (mm)	J11.2
k	Plate buckling coefficient	E7.1
k_v	Web plate shear buckling coefficient	G2.1
l	Actual length of end-loaded weld, in. (mm)	J2.2b
l	Length of connection, in. (mm)	Table D3.1
l_b	Length of bearing, in. (mm)	J11
l_{end}	Distance from the near side of the connecting branch 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)	K2.1
l_p	Projected length of the overlapping branch on the chord, in. (mm)	K2.1
l_1	Half of the distance between the center of the hole and the center of the adjacent hole or distance between the center of the hole and the edge of the material, in the direction of the force, in. (mm)	J3.10
l_1, l_2	Connection weld length, in. (mm)	Table D3.1
l_2	Half of the distance between the center of the hole and the center of the adjacent hole or distance between the center of the hole and the edge of the material, in the direction perpendicular to the force, in. (mm)	J3.10
n	Strain hardening coefficient	App. 7.1.1
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_{eff}	Auxiliary coefficient	C2.3
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
$n(T)$	Strain hardening coefficient at elevated temperatures	App. 7.2
p	Pitch, in. per thread (mm per thread)	App. 3.4
r	Radius of gyration, in. (mm)	E2
r	Internal radius of corner, in. (mm)	B4.3

Symbol	Definition	Section
r	Retention factor depending on bottom flange temperature	App. 4.2.4d
r_a	Radius of gyration about the geometric axis, in. (mm)	E5
r_i	Minimum 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, 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
s	Longitudinal center-to-center spacing (pitch) of any two consecutive holes, in. (mm)	B4.4b
t	Design wall thickness of round HSS, in. (mm)	B4.1b
t	Design wall thickness of HSS member, in. (mm)	Table D3.1
t	Design thickness of plate, in. (mm)	D5.1
t	Design thickness, in. (mm)	G4
t	Element design thickness, as defined in Section B4.2, in. (mm)	G6
t	Design thickness, as defined in Section B4.2, corresponding to longer side, in. (mm)	G8.2
t	Thickness of connected material, in. (mm)	J3.10
t	Total thickness of fillers, in. (mm)	J5.3
t	Design wall thickness of HSS main member, in. (mm)	K2.1
t	Distance from the neutral axis to the extreme tensile fibers, in. (mm)	App. 6.3.2a
t_b	Design wall thickness of HSS branch member, in. (mm)	K2.1
t_{cf}	Design thickness of column flange, in. (mm)	J11.6
t_f	Design thickness of flange, in. (mm)	B4.2b
t_f	Design thickness of the loaded flange, in. (mm)	J11.1
t_p	Design thickness of plate, in. (mm)	Table D3.1
t_p	Design thickness of tension loaded plate, in. (mm)	App. 3.3
t_{st}	Design thickness of web stiffener, in. (mm)	App. 6.3.2a
t_w	Design thickness of web, in. (mm)	B4.2b
t_w	Design thickness of column web, in. (mm)	J11.6
w	Size of weld leg, in. (mm)	J2.2b
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
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
$\overline{x}$	Eccentricity of connection, in. (mm)	Table D3.1
y	Subscript relating symbol to minor-axis bending	H1.1
Λ	Upper bound strain limit	App. 1.3.3d
α	Flexural buckling coefficient	E3
α	Bending coefficient, determined from Table A-2.2.1	App. 2.6
α_{csm}	CSM interaction coefficient for biaxial bending	App. 2.7.2
β	Length reduction factor	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 square HSS	K2.1

Symbol	Definition	Section
β_{csm}	CSM interaction coefficient for biaxial bending	App. 2.7.2
β_{br}	Required shear stiffness of the bracing system, kip/in. (N/mm)	App. 6.2.1a
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	K2.1
β_{LT}	Elastic lateral-torsional buckling reduction coefficients determined from Table F2.1	F2.2
$\beta_{p,LT}$, $\beta_{y,LT}$	Lateral-torsional buckling coefficient determined from Table F2.1	F2.2
β_{sec}	Web distortional stiffness, including the effect of web transverse stiffeners, kip-in./rad (N-mm/rad)	App. 6.3.2a
β_T	Overall brace system required stiffness, kip-in./rad (N-mm/rad)	App. 6.3.2a
β_o, β_1, β_2	Flexural buckling coefficients	E3
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 square HSS	K2.1
ε	Engineering strain	App. 7.1
ε_{corner}	Strain induced in the corner of the rectangular HSS	B4.3
ε_{csm}	CSM strain limit	App. 1.3.3d
ε_{csm}	Cross-section failure strain	App. 2.3.2
$\varepsilon_{csm,c}$	Maximum CSM compressive strain	App. 2.6
$\varepsilon_{csm,max}$	Maximum CSM design strain	App. 2.6
ε_f	Specified minimum elongation after rupture determined over a length of 2 in. (50 mm)	B4.3
ε_r	Required strain	App. 1.3.3
ε_{rnd}	Strain induced in round HSS	B4.3
ε_u	Ultimate strain	B4.3
ε_u	Strain at the ultimate tensile stress	App. 2.2
ε_{wall}	Strain induced in the flats of the rectangular HSS	B4.3
ε_y	Yield strain	App. 2.2
$\varepsilon(T)$	Engineering strain at elevated temperatures	App. 7.2
$\varepsilon_u(T)$	Ultimate strain at elevated temperatures	App. 7.2
$\varepsilon_y(T)$	Strain at the yield stress at elevated temperatures	App. 7.2
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	K2.1
ζ	Gap ratio; the ratio of the gap between the branches of a gapped K-connection to the width of the chord for square HSS	K2.1
λ	Width-to-thickness ratio for the element	E7.1
λ_l	Local cross-section slenderness	App. 1.3.3d
λ_l	Cross-section slenderness	App. 2.1
λ_{pf}	Limiting width-to-thickness ratio for compact flange	F7.2
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	F7.2
μ	Mean slip coefficient	J3.8
ν	Poisson's ratio = 0.3	E7.1
ρ_{csm}	Reduction factor	App. 1.3.3e

Symbol	Definition	Section
ρ_w	Maximum shear ratio within the web panels on each side of the transverse stiffener	G2.3
τ_b	General stiffness reduction factor	App. 1.2.2b
τ_g	General stiffness reduction factor	C2.3
ϕ	Resistance factor	B3.1
Ω	Safety factor	B3.2

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.

Alloy steel. A steel, other than a stainless steel, that conforms to the definition of alloy steel given in ASTM A941.

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

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).

Austenitic stainless steel. A stainless steel alloy that is predominantly face-centered cubic in structure and hardenable only by cold working.

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.

Base metal. Alloy being welded, brazed, soldered, or cut.

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†. In a connection, limit state of shear forces transmitted by the mechanical fastener to the connection elements.

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.

Bimetallic interface. Any location where stainless steel has a direct electrical contact to a dissimilar metal.

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

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.

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. Member fabricated from stainless steel components, which may include rolled or extruded sections, built-up sections, and/or plates, using intermittent welds or fasteners.

Built-up section (or shape). Section fabricated from stainless steel elements welded together with a continuous weld along the entire length of the member.

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

Carbon steel. A steel, other than stainless steel, that conforms to the definition of carbon steel given in ASTM A941.

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

Chemical descaling (pickling). Chemical descaling agents including aqueous solutions of sulfuric acid, or nitric and hydrofluoric acids in accordance with ASTM A380/A380M.

Chemical passivation. Chemical treatment of a stainless steel in accordance with ASTM A967/A967M with a mild oxidant, such as a nitric acid solution, for the purpose of the removal of free iron and other foreign matter, but which is generally not effective in removal of heat tint or oxide scale on stainless steel.

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

Cleaning. Removal of exogenous surface contamination, including dirt, grease, and free iron from contact with tools and other equipment, which may interfere with the formation of the passive metal oxide film.

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 stainless steel superstructure and the foundation.

Compact section. Section capable of developing a fully plastic stress distribution before the onset of local buckling.

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

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.

Continuous strength method (CSM). A deformation-based method that replaces the concept of cross-section classification with a continuous relationship between cross-section slenderness and deformation (strain) capacity, allowing for the incorporation of strain hardening.

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

Descaling. Removal of heavy, tightly adherent oxide films resulting from hot-forming, heat-treatment, welding, and other high-temperature operations.

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 Structural Stainless Steel Buildings* 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 thickness. Thickness assumed in the determination of section properties.

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

Diagonal stiffener. Web stiffener at the 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.

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

Distributed plasticity. Analysis in which the development and spread of plasticity through the depth of the cross section as well as along the length of the member is captured (see plastic zone).

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.

Duplex (austenitic-ferritic) stainless steel. A stainless steel alloy that is a mixture of austenitic and ferritic structures, with at least one-fourth of the lesser phase, and hardenable only by cold working.

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 that 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 that are in excess of the anticipated ambient conditions.

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, but it is not required to be. A combination of drawings and digital models also may be provided.

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, but it is not required to be. 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.

Filler metal. Alloy to be added to make a brazed, soldered, or 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 μin. (13 μm).

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 neglected.

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.

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.

Free iron. Oxidizable deposit from contact with iron, other steel alloy, or substance containing the element iron.

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

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

Galling (of threads). Displacement of material between mating threads during tightening that causes interface contact points to shear, producing high friction, increased resistance to tightening, or seizing of the threads.

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. 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.

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.

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

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

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.

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.

Joint root. Portion of a joint to be welded where the members approach closest to each other.

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.

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.

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.

*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. Heavy oxide layer on stainless steel formed during hot-forming, heat-treatment, welding, and other high-temperature operations.

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

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

Net tensile area. Minimum area of the threaded section of a bolt.

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 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.

Nominal thickness. Designated or theoretical thickness, as provided in tables for the relevant ASTM standard.

Noncompact section. Section that is able to develop the yield stress in its compression elements before local buckling occurs, but is unable to develop a fully plastic stress distribution.

Nondestructive testing. The process of determining acceptability of a material or a component in accordance with established criteria without impairing its future usefulness.

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.

Other steel alloys. Any steel alloy other than those listed in Section A3.1b, including carbon steel and alloy steel.

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.

Passivation. Treatment for corrosion-resistant steel to eliminate corrodible surface impurities and provide a protective film.

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 moment. Product of the strength at 0.2% offset permanent strain and the plastic section modulus.

Plastic zone. Analysis in which the development and spread of plasticity through the depth of the cross section as well as along the length of the member is captured (see *distributed plasticity*).

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.

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.

Procedure qualification records (PQR). Record of the actual variables used to weld a test coupon and results of required destructive and nondestructive tests.

Precipitation hardening stainless steel. A stainless steel alloy that may be basically austenitic or martensitic in structure and hardenable by precipitation hardening (sometimes called age hardening).

Prequalification. Requirements for exempting a welding procedure specification (WPS) from qualification by testing.

Prequalified welding procedure specification (PWPS). A welding procedure specification in compliance with the stipulated conditions of a particular welding code or specification and therefore acceptable for use under that code or specification without a requirement for qualification testing.

Pretensioned bolt. Bolt tightened to the specified minimum pretension.

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

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.

Qualification. Requirements for qualification of WPS and welding personnel (welders and welding operators) by testing, including the tests required and ranges qualified.

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.

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*.

Rotation capacity. Incremental angular rotation defined as the ratio of the inelastic rotation attained to the idealized elastic rotation at first yield prior to significant load shedding.

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.

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 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.

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

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. Cross section possessing plate components of sufficient slenderness such that local buckling in the elastic range will occur.

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.

Stainless steel. A steel that conforms to a specification that requires, by mass, a minimum chromium content of 10.5%, and a maximum carbon content of 1.20%.

Standard welding procedure specification (SWPS). Welding procedure specification qualified according to the requirements of AWS B2.1/AWS B2.1M, approved by AWS, and made available for production welding by companies or individuals other than those performing the qualification test.

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.

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

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

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 powder, plasma, or laser.

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 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 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.

UNS (unified numbering system) designation. Identification system for specific metals and alloys; stainless steel alloys are identified in the ASTM standards in accordance with ASTM E527 and SAE J1086.

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

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

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

Weldability. The capacity of material to be welded under the imposed fabrication conditions into a specific, suitably designed structure performing satisfactorily in the intended service.

Welding procedure specification (WPS). A document providing the required welding variables for a specific application to assure repeatability by properly trained welders and welding operators.

Weld metal. Metal in a fusion weld consisting of that portion of the base metal and filler metal melted during welding.

Weld root. See *joint root*.

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. Product of the strength at 0.2% offset permanent strain and the elastic section modulus.

Yield strength. Stress at which a material exhibits a specified limiting deviation from the proportionality of stress to strain as defined by ASTM A370, which is determined by the offset method for stainless steel where a plastic strain of 0.2% is specified.

Yield stress. Generic term to denote yield strength.

ABBREVIATIONS

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

ABC (applicable building code)

ACI (American Concrete Institute)

AHJ (authority having jurisdiction)

AISC (American Institute of Steel Construction)

AISI (American Iron and Steel Institute)

ANDE (ASME Nondestructive Examination)

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)

ASTM (ASTM International)

AWI (associate welding inspector)

AWS (American Welding Society)

CJP (complete joint penetration)

CSM (continuous strength method)

CVN (Charpy V-notch)

ENA (elastic neutral axis)

EOR (engineer of record)

ERW (electric resistance welding)

FCAW (flux cored arc welding)

FR (fully restrained)

GMAW (gas metal arc welding)

GTAW (gas tungsten arc welding)

HBW (Brinell hardness number obtained using a tungsten carbide ball indenter)

HRC (Rockwell hardness scale C)

HSLA (high-strength low-alloy)

HSS (hollow structural section)

ISO (International Organization for Standardization)

LRFD (load and resistance factor design)

MIC (microbiological influenced corrosion)

MT (magnetic particle testing)

NDT (nondestructive testing)

OSHA (Occupational Safety and Health Administration)

PJP (partial joint penetration)

PQR (procedure qualification record)

PR (partially restrained)

PT (penetrant testing)

PWPS (prequalified welding procedure specification)

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)

SDI (Steel Deck Institute)

SEI (Structural Engineering Institute)

SFPE (Society of Fire Protection Engineers)

SMAW (shielded metal arc welding)

SSPC (Society for Protective Coatings)

SWI (senior welding inspector)

SWPS (standard welding procedure specification)

UNC (unified national coarse)

UNS (unified numbering system)

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, products, and structural design documents.

The chapter is organized as follows:

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

User Note: The Commentary and AISC Design Guide 27, *Structural Stainless Steel*, give guidance on the design of stainless steel and how to select a stainless steel alloy for a range of service conditions.

A1. SCOPE

The *Specification for Structural Stainless Steel Buildings* (ANSI/AISC 370), hereafter referred to as this Specification, shall apply to the design, fabrication, and erection of structural stainless steel systems, where the stainless steel elements are defined in AISC *Code of Standard Practice for Structural Stainless Steel Buildings* (AISC 313) Section 2.1.

This Specification sets forth criteria for the design, fabrication, and erection of structural stainless steel buildings and other structures, including industrial 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. This Specification provides criteria for the design of the austenitic and duplex (austenitic-ferritic) stainless steel alloy families for structural shapes. Criteria are also given for the design of the precipitation hardening stainless steel alloy family for tension members, fittings, and fasteners.

User Note: Stainless steel alloys are typically specified based on the required corrosion resistance.

Wherever this Specification refers to the applicable building code and there is no code listed, 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: This Specification applies to stainless steel structural members in a range of product forms, including hollow structural sections. However, provisions for cold-formed stainless steel structural members are also available in the ASCE *Specification for the Design of Cold-Formed Stainless Steel Structural Members* (ASCE/SEI 8-21).

This Specification includes the Symbols, the Glossary, Abbreviations, Chapters A through N, and Appendices 1 through 7. 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.

1. Seismic Applications

The provisions of this standard do not address seismic applications.

User Note: This does not preclude the engineer of record (EOR) from employing structural stainless steel in seismic applications; however, the provisions of ASCE/SEI 7, Section 12.2.1.1, should be employed to justify and document selected seismic response modification coefficients.

2. Nuclear Applications

The design, fabrication, and erection of structural stainless steel when used in 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).

User Note: The current version of the AISC *Specification for Safety-Related Steel Structures for Nuclear Facilities*, ANSI/AISC N690-18, was written as a supplement to the AISC *Specification for Structural Steel Buildings*, ANSI/AISC 360-16. At the time of the development of AISC/ANSI N690-18, this Specification was not available. The intent of this Section is that the requirements in ANSI/AISC N690-18 supersede the requirements in this Specification for the type of stainless steels covered in this Specification when designing nuclear structures. The next version of AISC N690 (2024) will directly reference this Specification.

A2. REFERENCED SPECIFICATIONS, CODES, AND STANDARDS

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)

AISC 313-21 *Code of Standard Practice for Structural Stainless Steel Buildings*

ANSI/AISC N690-18 *Specification for Safety-Related Steel Structures for Nuclear Facilities*

(c) American Iron and Steel Institute (AISI)

AISI S902-17 *Test Standard for Determining the Effective Area of Cold-Formed Steel Compression Members*

(d) American Society of Civil Engineers (ASCE)

ASCE/SEI 7-16 *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*

(e) American Society of Mechanical Engineers (ASME)

ASME ANDE-1-2015 *Nondestructive Examination and Quality Control Central Qualification and Certification Program*

ASME B1.1-2019 *Unified Inch Screw Threads (UN, UNR, and UNJ Thread Forms)*

ASME B18.2.1-2012 *Square, Hex, Heavy Hex, and Askew Head Bolts and Hex, Heavy Hex, Hex Flange, Lobed Head, and Lag Screws*

ASME B18.2.2-2015 *Nuts for General Applications: Machine Screw Nuts, Hex, Square, Hex Flange, and Coupling Nuts*

ASME B18.21.1-2009 *Washers: Helical Spring-Lock, Tooth Lock, and Plain Washers*

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-19 *Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling*

A182/A182M-20 *Standard Specification for Forged or Rolled Alloy and Stainless Steel Pipe Flanges, Forged Fittings, and Valves and Parts for High-Temperature Service*

A193/A193M-20 *Standard Specification for Alloy-Steel and Stainless Steel Bolting for High Temperature or High Pressure Service and Other Special Purpose Applications*

A194/A194M-20a *Standard Specification for Carbon Steel, Alloy Steel, and Stainless Steel Nuts for Bolts for High Pressure or High Temperature Service, or Both*

A240/A240M-20a *Standard Specification for Chromium and Chromium-Nickel Stainless Steel Plate, Sheet, and Strip for Pressure Vessels and for General Applications*

A276/A276M-17 *Standard Specification for Stainless Steel Bars and Shapes*

A312/A312M-19 *Standard Specification for Seamless, Welded, and Heavily Cold Worked Austenitic Stainless Steel Pipes*

A320/A320M-18 *Standard Specification for Alloy-Steel and Stainless Steel Bolting for Low-Temperature Service*

A351/A351M-18e1 *Standard Specification for Castings, Austenitic, for Pressure Containing Parts*

A370-20 *Standard Test Methods and Definitions for Mechanical Testing of Steel Products*

A380/A380M-17 *Standard Practice for Cleaning, Descaling, and Passivation of Stainless Steel Parts, Equipment, and Systems*

A453/A453M-17 *Standard Specification for High-Temperature Bolting, with Expansion Coefficients Comparable to Austenitic Stainless Steels*

A473-19 *Standard Specification for Stainless Steel Forgings*

A479/A479M-20 *Standard Specification for Stainless Steel Bars and Shapes for Use in Boilers and Other Pressure Vessels*

A480/A480M-20a *Standard Specification for General Requirements for Flat-Rolled Stainless and Heat-Resisting Steel Plate, Sheet, and Strip*

A484/A484M-20b *Standard Specification for General Requirements for Stainless Steel Bars, Billets, and Forgings*

A554-16 *Standard Specification for Welded Stainless Steel Mechanical Tubing*

A564/A564M-19a *Standard Specification for Hot-Rolled and Cold-Finished Age-Hardening Stainless Steel Bars and Shapes*

A666/A666M-15 *Standard Specification for Annealed or Cold-Worked Austenitic Stainless Steel Sheet, Strip, Plate, and Flat Bar*

A673/A673M-17 *Standard Specification for Sampling Procedure for Impact Testing of Structural Steel*

A693-16 *Standard Specification for Precipitation-Hardening Stainless and Heat-Resisting Steel Plate, Sheet, and Strip*

A705/A705M-20 *Standard Specification for Age-Hardening Stainless Steel Forgings*

A747/A747M-18 *Standard Specification for Steel Castings, Stainless, Precipitation Hardening*

A751-20 *Standard Test Methods, Practices, and Terminology for Chemical Analysis of Steel Products*

A790/A790M-20 *Standard Specification for Seamless and Welded Ferritic/Austenitic Stainless Steel Pipe*

A890/A890M-18a *Standard Specification for Castings, Iron Chromium Nickel Molybdenum Corrosion Resistant, Duplex (Austenitic/Ferritic), for General Application*

A941-18 *Standard Terminology Relating to Steel, Stainless Steel, Related Alloys, and Ferroalloys*

A962/A962M-19 *Standard Specification for Common Requirements for Bolting Intended for Use at any Temperature from Cryogenic to the Creep Range*

A967/A967M-17 *Standard Specification for Chemical Passivation Treatments for Stainless Steel Parts*

A999/A999M-18 *Standard Specification for General Requirements of Alloy and Stainless Steel Pipe*

A1049/A1049M-18 *Standard Specification for Stainless Steel Forgings, Ferritic/Austenitic (Duplex), for Pressure Vessels and Related Components*

A1069/A1069M-19 *Standard Specification for Laser and Laser Hybrid Welded Stainless Steel Bars, Plates, and Shapes*

A1082/A1082M-16 *Standard Specification for High Strength Precipitation Hardening and Duplex Stainless Steel Bolting for Special Purpose Applications*

D4417-20a *Standard Test Methods for Field Meaurement of Surface Profile of Blast Cleaned Steel*

D7127-17 *Standard Test Method for Measurement of Surface Roughness of Abrasive Blast Cleaned Metal Surfaces Using a Portable Stylus Instrument*

E119-20 *Standard Test Methods for Fire Tests of Building Construction and Materials*

E527-16 *Standard Practice for Numbering Metals and Alloys in the Unified Numbering System (UNS)*

F436/F436M-19 *Standard Specification for Hardened Steel Washers Inch and Metric Dimensions*

F593-17 *Standard Specification for Stainless Steel Bolts, Hex Cap Screws, and Studs*

F594-09(2020) *Standard Specification for Stainless Steel Nuts*

F606/F606M-19 *Standard Test Methods for Determining the Mechanical Properties of Externally and Internally Threaded Fasteners, Washers, Direct Tension Indicators, and Rivets*

F836M-20 *Standard Specification for Style 1 Stainless Steel Metric Nuts*

(h) American Welding Society (AWS)

AWS B2.1/B2.1M:2014-AMD1 *Specification for Welding Procedure and Performance Qualification*

AWS B5.1:2013-AMD1 *Specification for the Qualification of Welding Inspectors*

AWS D1.1/D1.1M:2020 *Structural Welding Code—Steel*

AWS D1.6/D1.6M:2017 *Structural Welding Code—Stainless Steel*

(i) International Organization for Standardization (ISO)

ISO 9712-2012 *Nondestructive Testing—Qualification and Certification of NDT Personnel*

(j) Research Council on Structural Connections (RCSC)

Specification for Structural Joints Using High-Strength Bolts, 2014

(k) SAE Standard
J1086-2012 *Practice for Numbering Metals and Alloys (UNS)*

(l) Society for Protective Coatings (SSPC)
SSPC PA 17 (2012) *Procedure for Determining Conformance to Steel Profile/Surface Roughness/Peak Count Requirements*

(m) Steel Deck Institute (SDI)
QA/QC-2017 *Standard for Quality Control and Quality Assurance for Installation of Steel Deck*

A3. MATERIAL AND PRODUCT ORDER REQUIREMENTS

1. Structural Stainless Steel Materials

An appropriate stainless steel shall be selected which, in addition to strength, takes into account factors including corrosion resistance suitable for the service environment, design details, surface finish, availability, fabrication, and maintenance requirements.

Material test reports and certifications shall comply with the requirements of the relevant ASTM standard, standard practice, or the standard method(s), and any other requirements in the order.

User Note: Stainless steel alloys are identified by the unified numbering system (UNS) designation in accordance with SAE J1086 and ASTM E527.

1a. Service Environment Assessment

The service environment shall be assessed prior to design to determine appropriate candidate alloys.

User Note: This section is not intended to create or imply a contractual requirement for the EOR responsible for the structural design, but should be conducted by a member of the design team or a designated consultant as determined by contractual agreement. In many cases, it is best for a metallurgical engineer with specific expertise in stainless steel to assist in selecting appropriate alloy(s) for the intended applications and performance.

1b. ASTM Standards and Minimum Order Requirements

The following stainless steel alloys in accordance with the ASTM standards in Table A3.1 are approved for use as structural components under this Specification:

Alloy Family:	Unified Numbering System (UNS) Designation:
Austenitic stainless steels	S30400, S30403, S31600, S31603, S31703, S32100, N08904, S31254, N08367, N08926

Duplex stainless steels	S32003, S32101, S32202, S32205, S32304, S32750, S32760, S82011, S82441
Precipitation hardening stainless steels	S15500, S17400

User Note: The UNS designations for the low carbon versions of the austenitic stainless steels S30400 (304) and S31600 (316) are S30403 (304L) and S31603 (316L). Specification of the lower carbon or dual certified version of these austenitics is recommended to reduce the risk of sensitization (chromium carbide precipitation to the grain boundaries) in the heat effected zones adjoining each weld. The other austenitic and duplex stainless steels included in this Specification are low carbon and have no special specification requirement.

User Note: Within the stainless steel industry, the terms “grade” and “type” are commonly used to mean base metal “alloy,” not strength level.

User Note: The Commentary gives specified minimum mechanical properties for stainless steel products according to the relevant ASTM standards.

When ordering stainless steel products, the order requirements given in Table A3.1 shall be followed. Finish, chemistry, product dimensions, special testing, special tolerances, condition, and other mandatory or special ordering requirements of the ASTM standards shall be specified as indicated for each product. The nominal thickness shall be specified in inches or millimeters.

User Note: ASTM standards do not define stainless steel gage thicknesses. While tables with typical industry gage thicknesses exist, they are not legally binding and individual producers may define gage thicknesses differently. The typical gage thicknesses associated with stainless steels are different from those for other steel alloys.

It is permitted to specify dimensional tolerances tighter than those listed in the relevant ASTM standards, and other special or supplementary requirements.

User Note: Any special and/or supplementary requirements that go beyond the minimum requirements of the relevant ASTM standards should be agreed contractually with the supplier and may limit material availability, lengthen delivery times, and increase costs and minimum order requirements.

TABLE A3.1
Order Requirements for Stainless Steel Products

(a) Hot-Rolled and Extruded Structural Shapes (Austenitic or Duplex)

Alloy Chemical Requirements	UNS designation
Condition	ASTM A276/A276M or A479/A479M
Finish	ASTM A484/A484M
Special or Supplementary Requirements	ASTM A276/A276M, A479/A479M, or A484/A484M

Notes:
1. Condition (heat treatment or processing) determines mechanical properties.
2. Minimum mechanical and chemical requirements are in ASTM A276/276M or A479/A479M.

(b) Hollow Structural Sections (Austenitic or Duplex)

Product Form	Round, square, rectangular hollow structural sections: ASTM A554	Welded round pipe: ASTM A312/312M (austenitic), ASTM A790/790M (duplex)
Alloy Chemical Requirements	UNS designation or ASTM A554 grade	UNS designation
Condition	ASTM A554	–
Finish	ASTM A554	ASTM A999/A999M
Special or Supplementary Requirements	ASTM A554	ASTM A312/A312M or ASTM A999/A999M (austenitic), A790/A790M (duplex)

Notes:
1. Minimum mechanical and chemical requirements are in ASTM A554, A312/A312M, or A790/A790M.
2. ASTM A312/A312M and ASTM A790/A790M cover both seamless and welded pipe; welded pipe should be specified for structural applications.
3. For duplex stainless steels specified to ASTM A554 manufactured without filler metal, a special requirement of heat treatment may be specified if the corrosion resistance of the weld needs to match that of the base metal. ASTM A790/A790M requires heat treatment unless otherwise specified.

(c) Built-Up Shapes—Laser and Laser Hybrid (Austenitic or Duplex)

Product Form	I-shapes, channels, angles, tees, box sections
Alloy Chemical Requirements	UNS designation or ASTM A554 grade (box sections)
Mechanical Properties	Grade 1: ASTM A240/A240M, A276/A276M, or A479/A479M Grades 2, 3, or 4: ASTM A1069/A1069M
Condition	ASTM 1069/A1069M
Finish	ASTM 1069/A1069M
Special and Supplementary Requirements	ASTM 1069/A1069M

Notes:
1. ASTM A1069/A1069M uses the dimensional tolerance requirements of ASTM A554 (box sections), and ASTM A6/A6M (built-up open or custom sections).
2. Specification of AWS D1.1/D1.1M, clause 7.22, member dimensional tolerances shall be a special ordering requirement, as applicable.
3. Minimum chemical requirements are in ASTM A240/A240M, A276/A276M, or A479/A479M.

TABLE A3.1 (continued)
Order Requirements for Stainless Steel Products

(d) Built-Up Shapes—Other Than Laser and Laser Hybrid (Austenitic or Duplex)[a]

Product Form	I-shapes, channels, angles, tees, box sections
Weld Filler Material Requirements	Weld strength and corrosion resistance
Dimensional Tolerance	Open Shapes • ASTM A6/A6M for member tolerances of built-up shapes • Specification of AWS D1.1/D1.1M, clause 7.22, member dimensional tolerances shall be a special ordering requirement, as applicable Box Sections • ASTM A554, or as specified in design documents
Profile and Welding Requirements	As shown in design documents
Alloy Chemical Requirements	UNS designation
Base Metal	Table A3.1 (a) and (e)
Weld Mechanical Properties	As shown in design documents
Welding and Inspection	AWS D1.6/D1.6M
Finish, Supplementary Requirements	ASTM A1069/A1069M or ASTM A484/A484M
Cleaning/Weld Restoration	ASTM A380/A380M

[a] **User Note:** Documentation for all purchased built-up shapes should comply with the requirements in Section N3.2 and the requirements of Table A3.1(d). Acceptable documentation should consist of (1) mill certification to the requirements listed in Table A3.1(d), or (2) a statement of compliance with the requirements of Table A3.1(d) on company letterhead, including copies of the documentation required in Section N3.2.

Notes:
1. The minimum mechanical and chemical requirements for the base material are in ASTM A240/A240M, A276/A276M, or A479/A479M.
2. Specify whether complete-joint-penetration groove welds are required.
3. If the strength of the base metal has been increased by cold work, the base metal around the weld and standard filler metals will not have matching strength. Specialized filler metals can be used but they can increase costs.
4. Heat treatment may be required for matching weld corrosion resistance, particularly if duplex stainless steels are welded without filler metal.
5. Built-up shapes made by a fabricator shall be subject to the requirements of Chapter M and N. Requirements from Chapter M and N for built-up shapes made by a manufacturer shall be defined in the ordering requirements.
6. An AWS standard welding procedure specification (SWPS) based on AWS B2.1/B2.1M (AWS B2.1-X-XXX series) is permitted.
7. It is permitted to use ASTM A1069/A1069M for marking, packaging, and other requirements that are not related to product manufacture.

(e) Plate, Sheet, Strip (Austenitic or Duplex)

Product Form	Sheet, strip, or plate
Alloy Chemical Requirements	UNS designation
Condition	ASTM A480/A480M
Finish	ASTM A480/A480M
Special or Supplementary Requirements	ASTM A240/A240M or ASTM A480/A480M

Note: Minimum mechanical and chemical requirements are in ASTM A240/A240M.

TABLE A3.1 (continued)
Order Requirements for Stainless Steel Products

(f) Hot- and Cold-Finished Bar and Flat Bar Cut from Strip or Plate

	Austenitic or Duplex	Precipitation Hardening
Product Form	Round, square, or hexagonal bar Flat bar cut from plate	Round, square, or hexagonal bar
Alloy Chemical Requirements	UNS designation	UNS designation
Condition	ASTM A276/A276M	ASTM A564/A564M
Finish	ASTM A484/A484M	ASTM A484/A484M
Special or Supplementary Requirements	ASTM A484/A484M ASTM A276/A276M	ASTM A484/A484M or ASTM A564/A564M

Note: Minimum mechanical and chemical requirements are in ASTM A276/A276M, A479/A479M, or A564/A564M.

General Notes:
1. Dimensions shall be specified.
2. Special requirements for chemistry, testing, tolerances, marking, shipping, or packaging shall be specified when needed. Magnetic permeability may be specified for austenitic stainless steels only.
3. ASTM A479/A479M includes supplementary requirements for high temperature service (S1), intergranular corrosion testing (S2), high cycle fatigue service (S4), and optimal resistance to chloride stress corrosion cracking (S5).

User Note: Table User Note A3.1 lists some physical properties of stainless steels.

TABLE USER NOTE A3.1
Physical Properties of Stainless Steels

Alloy Family	Modulus of Elasticity, *E*		Shear Modulus of Elasticity, *G*		Density		Coefficient of Thermal Expansion between 68 and 212°F (20 and 100°C)	
	ksi	MPa	ksi	MPa	lb/ft^3	kg/m^3	$10^{-6}/°F$	$10^{-6}/°C$
Austenitic stainless steel[a]	28,000	193 000	10,800	74 500	500	8 000	9.1	16
Duplex stainless steel	29,000	200 000	11,200	77 200	485	7 800	7.4	13
Precipitation hardening stainless steel	28,500	197 000	11,000	75 800	485	7 800	6.1	11

[a] For cold-finished austenitic bar in Condition B (relatively severe cold work) in accordance with ASTM A276/A276M, the modulus of elasticity = 25,000 ksi (172 000 MPa) and shear modulus of elasticity = 9,600 ksi (66 000 MPa).

1c. Built-Up Heavy Shapes

Built-up cross sections consisting of duplex stainless steel plates with a thickness exceeding 2 in. (50 mm) are considered built-up heavy shapes. Built-up heavy shapes used as members subject to 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 in accordance with this section. The structural design documents shall require that the steel be supplied with Charpy V-notch impact test results, including the frequency of testing, the test temperature to be used, and the absorbed energy requirements. 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:** Impact tests are not required for austenitic stainless steels because they are not susceptible to brittle rupture, even at low temperatures. They demonstrate impact toughness well above 74 ft-lb (100 J) at –320°F (–196°C).

2. Castings and Forgings

Stainless steel castings and forgings shall conform to an ASTM standard and shall provide strength, ductility, weldability, and toughness adequate for the purpose. Test reports produced in accordance with the ASTM reference standards or standard test methods shall constitute sufficient evidence of conformity with such standards. The following specifications apply:

Castings
ASTM A351/A351M (austenitic stainless steel)
ASTM A747/A747M (precipitation hardening stainless steel)
ASTM A890/A890M (duplex stainless steel)

Forgings
ASTM A182/A182M
ASTM A473
ASTM A705/A705M (precipitation hardening stainless steel)
ASTM A1049/A1049M (duplex stainless steel)

3. Bolts, Washers, and Nuts

The bolts, washers, and nuts shall all have equivalent or greater corrosion resistance than the most corrosion resistant of the metal alloys joined.

Bolt, washer, and nut material conforming to one of the following ASTM standards is approved for use under this Specification. In addition to the alloys listed in Section A3.1b, precipitation hardening stainless steel S66286 is also approved for use.

(a) Bolts
ASTM A193/A193M
ASTM A320/A320M
ASTM A453/A453M
ASTM A1082/A1082M
ASTM F593

(b) Nuts
ASTM A194/A194M
ASTM A453/A453M
ASTM A962/A962M
ASTM F594
ASTM F836M

(c) Washers
The chemical composition and mechanical properties of the stainless steel washer raw material shall meet the requirements of ASTM A240/A240M, ASTM A453/A453M, ASTM A666/A666M, or ASTM A693. Unless otherwise specified, the requirements for hardness testing of ASTM F436/F436M shall apply.

User Note: There are no ASTM standards for stainless steel washers.

Stainless steel washers hardened to at least 290 Brinell HBW (31 Rockwell HRC) shall be used under both the bolt head and nut in pretensioned joints subject to fatigue loading, and slip-critical joints.

Surface hardening shall conform to the requirements of Section A3.5.

User Note: It is good practice to use hardened stainless steel washers under both the bolt head and nut in all pretensioned joints, not only in joints subject to fatigue loading.

User Note: Other stainless steel or nonferrous alloys may be suitable for bolts, washers, and nuts in specific applications, but the evaluation of those alloys is the responsibility of the engineer specifying them.

User Note: Care is required for the use of S15500 and S17400 in structurally critical applications. In H1150 heat conditions, these alloys present a lower risk of hydrogen embrittlement and stress corrosion cracking than in the other heat treatment conditions. Precipitation hardening stainless steel S66286 is more resistant to hydrogen embrittlement and stress corrosion cracking than S15500 and S17400.

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

4. Anchor Rods and Threaded Rods

Anchor rod and threaded rod made of austenitic, duplex, or precipitation hardening stainless steel material shall conform to one of the following ASTM standards:

ASTM A193/A193M
ASTM A320/A320M
ASTM A1082/A1082M
ASTM F593

Threads on anchor rods and threaded rods shall conform to the Unified Standard Series of ASME B18.2.1 and shall have Class 2A tolerances.

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

5. Surface Hardening

The following methods for surface hardening of stainless steel are permitted at a temperature below 850°F (450°C):

(a) Nitrocarburizing or low temperature carburizing of austenitic stainless steels

> **User Note:** Nitriding of austenitic stainless steels is discouraged because it produces a very thin hardened layer that may not provide adequate galling or wear protection, and it reduces the alloy's corrosion resistance. If this method is used, the austenitic stainless steel alloy should be low carbon and in the annealed condition prior to nitriding, and the effect of the reduction in corrosion resistance on performance in the service environment should be carefully evaluated.

(b) Low-temperature carburizing of duplex stainless steel

(c) Nitriding, nitrocarburizing, or low-temperature carburizing of precipitation-hardening stainless steels

Hardness testing of materials other than washers shall be in conformance with ASTM A370.

6. Consumables for Welding

Filler and base metal combinations shall be in accordance with the prequalified materials listed in AWS D1.6/D1.6M, clause 5, or a filler and base metal combination documented to meet the structural and corrosion performance requirements of the application. AWS D1.6/D1.6M, clause 6, shall be used to qualify all filler and base material combinations that do not meet the prequalification requirements of AWS D1.6/D1.6M, clause 5. Manufacturer's certification shall constitute sufficient evidence of conformity with the standards.

User Note: Options for filler and base metal combinations, based on minimum cost and strength considerations and not corrosion or service requirements, are given in the informative Annex D of AWS D1.6/D1.6M.

A4. MINIMUM ASSESSMENT REQUIREMENTS FOR SPECIFYING ALLOY CORROSION RESISTANCE

Minimum assessment requirements for specifying corrosion resistance are established in this section. A suitable stainless steel alloy(s) shall be selected for the service environment.

1. Galvanic Corrosion

Whenever dissimilar metals are in direct contact in any location, the potential for galvanic corrosion shall be assessed. If isolation is required by the assessment, the means of isolation shall be specified.

2. Chloride Stress Corrosion Cracking

Structural members exposed to chloride salts shall be assessed for their potential for chloride stress corrosion cracking.

User Note: Common austenitic stainless steels (i.e., S30400/S30403, S31600/S31603) are susceptible to chloride stress corrosion cracking. Service environments potentially prone to chloride stress corrosion cracking include, but are not limited to, indoor swimming pool areas, certain industrial and food processing environments, the interface between air and brackish or sea water, and structural members in areas with high accumulation of coastal or deicing salts.

3. Crevice Corrosion

Connections, assemblies, and the areas where surface deposits may accumulate shall be assessed for their crevice corrosion potential.

4. Microbiological Corrosion

When structural members are exposed to water or moist soil where microbiological organisms are expected to be present, the design and service environment shall be assessed for the likelihood of microbiologically influenced corrosion (MIC), also known as biocorrosion. In addition to specifying a suitable stainless steel alloy(s) for the service environment, design or operation modifications shall be made to address the risk.

5. Corrosion Caused by Immersion or Splashing by Liquids

Structural members that are immersed in liquids on a continuous, cyclical, or frequent basis, or regularly splashed by them, shall be assessed for their corrosion susceptibility.

A5. STRUCTURAL DESIGN DOCUMENTS AND SPECIFICATIONS

The structural design documents and specifications shall meet the requirements of the *Code of Standard Practice for Structural Stainless Steel Buildings*.

User Note: Provisions in this Specification contain information that is to be shown on design documents. These include:

- Section A3: UNS alloy designation for all components
- Section A3: Finish requirements
- Section A3.1c: Built-up heavy shapes where CVN toughness is required
- Section J3.1: Locations of connections using pretensioned bolts

Other information needed by the fabricator or erector should be shown on design documents, including:

- Packaging, handing, and storage requirements
- Post installation cleaning and finish restoration requirements, as required
- Weld cleaning requirements
- Weld corrosion testing, if required
- Post fabrication passivation, if required
- Fatigue details requiring nondestructive testing
- Risk category (Chapter N)
- Indication of complete-joint-penetration (CJP) groove welds subject to tension (Chapter N)

CHAPTER B
DESIGN REQUIREMENTS

This chapter addresses general requirements for the design of austenitic and duplex stainless steel structures applicable to all chapters of this Specification, and for tension members, fittings, and fasteners consisting of precipitation hardening stainless steel.

The chapter is organized as follows:

B1. General Provisions
B2. Loads and Load Combinations
B3. Design Basis
B4. Member Properties
B5. Fabrication and Erection
B6. Quality Control and Quality Assurance
B7. Evaluation of Existing Structures

B1. GENERAL PROVISIONS

The design of members and connections shall be consistent with the intended behavior of the framing 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 ASCE *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 subject 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).

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:** AISC *Code of Standard Practice for Structural Stainless Steel Buildings* 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 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

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.

11. Design for Fatigue

Fatigue shall be considered in accordance with Appendix 3, for members and their connections subject to repeated loading. Fatigue need not be considered 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.

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

B4. MEMBER PROPERTIES

User Note: If the production route for the section is not known at the time of design, it is generally conservative to assume the section is built up when computing the cross-sectional properties required for strength verification in accordance with Chapters E through K and Appendices 1, 2, and 4. However, if a hot-rolled section with tapered flanges is subsequently selected, the assumption that the flange thickness of the section is constant will lead to an overestimation of the minor axis and torsional cross-sectional 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.

The section classification of rectangular and round hollow structural sections (HSS) that are designed on the basis of the average yield strength, $F_{y,avg}$, as determined in Section B4.3, shall be based on the average yield strength by substituting $F_{y,avg}$ for

F_y when computing the limiting width-to-thickness ratios from Table B4.1a or Table B4.1b.

User Note: The value of F_y used in the determination of the limiting width-to-thickness ratios given in Tables B4.1a and B4.1b can be obtained from the relevant ASTM standard or the Commentary on Section A3.

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 or members, 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 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 as defined in Section B4.2a.

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

(e) For round hollow structural sections (HSS), the width shall be taken as the diameter, D.

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

User Note: If the production route for the section is not known at the time of design, h should conservatively be taken as the clear distance between flanges.

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 Thickness

2a. Design Thickness for Built-Up, Hot-Rolled, and Flat Bar Sections

The design thickness—t_f for flanges; t_w for webs; and t for box sections, plates, or flat bars—shall be used in calculations involving the thickness of built-up, hot-rolled, plate, or flat bar sections. The design thickness shall be taken equal to the nominal thickness for thicknesses greater than 3⁄16 in. (5 mm). For thicknesses of any element of the section less than or equal to 3⁄16 in. (5 mm), the design thickness of the element shall be taken equal to 0.95 times the nominal thickness. For all sections with a minimum thickness tolerance less than 0.05 times the nominal thickness, it is permitted to use the nominal thickness for the design thickness.

2b. 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 0.95 times the nominal wall thickness unless the section has a minimum thickness tolerance less than 0.05 times the nominal wall thickness, in which case it is permitted to use the nominal wall thickness for the design 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 one of the ASTM specifications in Table A3.1, and the appropriate limitations of this Specification are used.

3. Strength Increase in Stainless Steel HSS from Cold Forming

Strength increase from cold work in stainless steel HSS manufactured by cold forming is permitted by substituting $F_{y,avg}$ for F_y, where $F_{y,avg}$ is the average yield stress of the full section. Such increase shall be limited to Chapters D, E, F, and H, and Appendix 2. The limits and methods for determining $F_{y,avg}$ shall be in accordance with (a) and (b).

(a) The average yield stress, $F_{y,avg}$, of stainless steel cold-formed HSS shall be determined on the basis of one of the following methods:

(1) Full section tensile tests in accordance with ASTM A370, or

(2) Stub column tests in accordance with AISI S902, or

(3) Computed in accordance with Equation B4-1a or B4-1b:

(a) For cold-formed rectangular HSS

$$F_{y,avg} = \frac{F_{y,corner}A_{corner} + F_{y,wall}\left(A_g - A_{corner}\right)}{A_g} \le F_u \tag{B4-1a}$$

(b) For cold-formed round HSS

$$F_{y,avg} = F_{y,rnd} \tag{B4-1b}$$

where

A_{corner} = total corner area including a region of length $2t$ extending around the perimeter of the cross section on both sides of each corner, in.^{2} (mm^2)

$$= (\pi t)(2r+t)+16t^2 \tag{B4-2}$$

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

$F_{y,corner}$ = tensile yield stress of corners of cold-formed rectangular HSS, ksi (MPa)

$$= 0.85\frac{F_y}{\varepsilon_y^n}\left(\varepsilon_{corner} + \varepsilon_y\right)^n \text{ and } F_y \le F_{y,corner} \le F_u \tag{B4-3}$$

$F_{y,rnd}$ = tensile yield stress of cold-formed round HSS, ksi (MPa)

$$= 0.85\frac{F_y}{\varepsilon_y^n}\left(\varepsilon_{rnd} + \varepsilon_y\right)^n \text{ and } F_y \le F_{y,rnd} \le F_u \tag{B4-4}$$

$F_{y,wall}$ = tensile yield stress of the flats of cold-formed rectangular HSS, ksi (MPa)

$$= 0.85\frac{F_y}{\varepsilon_y^n}\left(\varepsilon_{wall} + \varepsilon_y\right)^n \text{ and } F_y \le F_{y,wall} \le F_u \tag{B4-5}$$

$$n = \frac{\log\left(F_y/F_u\right)}{\log\left(\varepsilon_y/\varepsilon_u\right)} \tag{B4-6}$$

ε_{corner} = strain induced in the corner of cold-formed rectangular HSS

$$= \frac{t}{2(2r+t)} \tag{B4-7}$$

ε_{rnd} = strain induced in cold-formed round HSS

$$= \frac{t}{2(D-t)} \tag{B4-8}$$

ε_u = ultimate strain, which may be approximated according to Equation B4-9 if not known

$$= 1-\frac{F_y}{F_u} \le \varepsilon_f \tag{B4-9}$$

ε_f = specified minimum elongation after rupture determined over a length of 2 in. (50 mm)

ε_{wall} = strain induced in the flats of cold-formed rectangular HSS

$$= \frac{t}{35.43} + \frac{\pi t}{2\left(B + H - 2t\right)} \quad \text{(B4-10)}$$

$$= \frac{t}{900} + \frac{\pi t}{2\left(B + H - 2t\right)} \quad \text{(B4-10M)}$$

$$\varepsilon_y = 0.002 + \frac{F_y}{E} \quad \text{(B4-11)}$$

B = overall width of square and rectangular HSS, in. (mm)
D = outside diameter of round HSS, in. (mm)
E = modulus of elasticity of stainless steel, ksi (MPa)
= 28,000 ksi (193 000 MPa) for austenitic, and 29,000 ksi (200 000 MPa) for duplex stainless steel
F_u = specified minimum tensile strength, ksi (MPa)
F_y = specified minimum yield stress, ksi (MPa)
H = overall height of square and rectangular HSS, in. (mm)
r = internal radius of corner, which may be taken as $2t$ if not known, in. (mm)

(b) The effect of any welding on mechanical properties of a cold-formed structural stainless steel member shall be determined on the basis of tests of full-section specimens containing, within the gage length, such welding as the manufacturer intends to use. Any necessary allowance for such effect shall be made in the structural use of the member.

User Note: Welding of the structural section may lead to annealing and hence a localized reduction in the strength increase from cold forming. It is therefore recommended that the strength at cross sections where the member is welded to other components be based on the specified minimum yield stress, F_y, unless test data justify the use of a higher value.

4. Gross and Net Area Determination

4a. Gross Area

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

4b. 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 and erection documents shall satisfy the requirements stipulated in Chapter M.

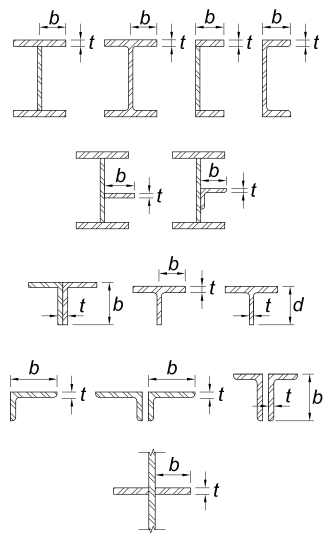
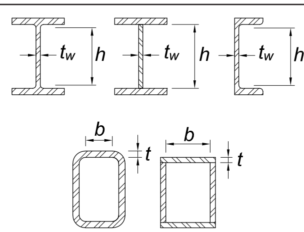
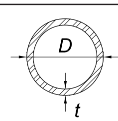
B6. QUALITY CONTROL AND QUALITY ASSURANCE

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

B7. EVALUATION OF EXISTING STRUCTURES

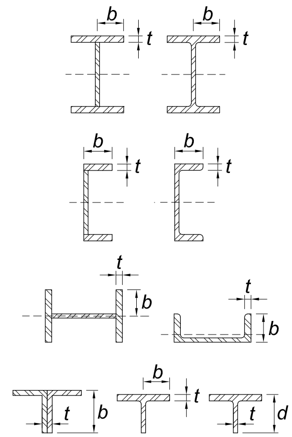
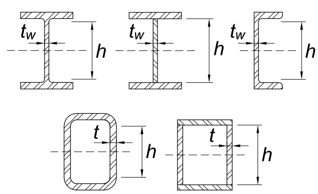
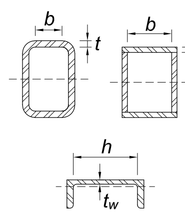
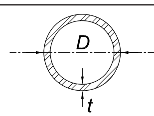
The evaluation of existing structures shall satisfy the requirements stipulated in Appendix 5.

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 built-up or rolled I-shaped sections and channels, plates, or angle legs projecting from built-up or rolled I-shaped sections, outstanding legs of pairs of angles connected with continuous contact, flanges and stems of tees, legs of single angles, legs of double angles with separators, and all other unstiffened elements	b/t or d/t	$0.41\sqrt{\dfrac{E}{F_y}}$	
Stiffened Elements	2	Webs of built-up or rolled I-shaped sections and channels, and walls of rectangular HSS and box sections of uniform thickness	h/t_w or b/t	$1.24\sqrt{\dfrac{E}{F_y}}$	
	3	Round HSS	D/t	$0.10\dfrac{E}{F_y}$	

E = modulus of elasticity of stainless steel, ksi (MPa)
= 28,000 ksi (193 000 MPa) for austenitic stainless steel, and 29,000 ksi (200 000 MPa) for duplex stainless steels
F_y = specified minimum yield stress, ksi (MPa), given in the relevant ASTM standard

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 Ratios: λ_p (compact/ noncompact)	Limiting Width-to-Thickness Ratios: λ_r (non-compact/ slender)	Examples
Unstiffened Elements	4	Flanges of built-up or rolled I-shaped sections and channels, outstanding legs of pairs of angles connected with continuous contact, flanges and stems of tees, and all other unstiffened elements of sections in flexure about the major or minor axis	b/t	$0.33\sqrt{\dfrac{E}{F_y}}$	$0.41\sqrt{\dfrac{E}{F_y}}$	
Stiffened Elements	5	Webs of built-up or rolled I-shaped sections, channels, rectangular HSS, and box sections of uniform thickness	h/t_w or h/t	$2.54\sqrt{\dfrac{E}{F_y}}$	$3.01\sqrt{\dfrac{E}{F_y}}$	
	6	Flanges of rectangular HSS and box sections of uniform thickness, and webs of built-up or rolled channels in flexure about the minor axis	b/t or h/t_w	$1.17\sqrt{\dfrac{E}{F_y}}$	$1.24\sqrt{\dfrac{E}{F_y}}$	
	7	Round HSS	D/t	$0.08\dfrac{E}{F_y}$	$0.31\dfrac{E}{F_y}$	

E = modulus of elasticity of stainless steel, ksi (MPa)
= 28,000 ksi (193 000 MPa) for austenitic stainless steel, and 29,000 ksi (200 000 MPa) for duplex stainless steels
F_y = specified minimum yield stress, ksi (MPa), given in the relevant ASTM standard

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:

User Note: Alternative methods for the design of structures for stability are provided in Appendix 1 that allow for considering member imperfections and/or inelasticity directly within the analysis and may be particularly useful for more complex structures.

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 that 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.

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.

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.

(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 AISC *Code of Standard Practice for Structural Stainless Steel Buildings.* 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 AISC *Code of Standard Practice for Structural Stainless Steel Buildings* 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)
= 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 AISC *Code of Standard Practice for Structural Stainless Steel Buildings*. 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 general stiffness reduction factor, τ_g, given by Table C2.1 for different member types and axis of buckling 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.

$$\tau_b = \frac{1.0}{1.0 + 0.002 n_{eff} \dfrac{E}{F_y} \left(\dfrac{\alpha P_r}{P_{ns}} \right)^{(n_{eff} - 1)}} \tag{C2-2}$$

where

α = 1.0 (LRFD)

= 1.6 (ASD)

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)

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

n_{eff} = auxiliary coefficient given in Table C2.1, where n is the strain hardening coefficient given in Appendix 7.

> **User Note**: Taken together, Sections (a) and (b) require the use of $\tau_g \tau_b$ multiplied by the nominal elastic flexural stiffness, and τ_g multiplied by other nominal elastic stiffnesses, for structural stainless steel members in the analysis.

Table C2.1
τ Function Coefficients

Member Type	τ_g	n_{eff}
Rolled or built-up I-shaped sections buckling about the minor axis, and other sections not specified in this table	0.70	$0.45n$
Rolled or built-up I-shaped sections buckling about the major axis, welded box sections, and round HSS	0.70	$0.55n$
Rectangular HSS	0.70	n

(c) In structures to which Section C2.2b is applicable, it is permitted to use $\tau_b = 1.0$ for all noncomposite members if a notional load of $0.002Y_i$, where Y_i is defined in Section C2.2b(a), is applied at all levels in the direction that provides the greatest destabilizing effect, on all load combinations. These additional loads shall be added to those, if any, used to represent the effects of initial system 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 stainless 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 austenitic and duplex stainless steel members and precipitation hardening stainless steel rods subject to axial tension.

The chapter is organized as follows:

D1. Slenderness Limitations
D2. Tensile Strength
D3. Effective Net Area
D4. Built-Up Members
D5. Pin-Connected Members

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

User Note: The design of eyebars is outside the scope of this chapter.

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 austenitic and duplex stainless steel 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.

The design tensile strength, $\phi_t P_n$, and the allowable tensile strength, P_n/Ω_t, of an unthreaded tension rod of precipitation hardening stainless steel in accordance with ASTM A564/A564M shall be determined in accordance with the limit state of tensile yielding in the gross section. If the ends of the rod are threaded, the available tensile strength of the threaded portion shall also be checked according to Section J3.6.

(a) For tensile yielding in the gross section

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

For austenitic and duplex stainless steel tension members

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

For precipitation hardening stainless steel tension rods

$$\phi_t = 0.80 \text{ (LRFD)} \qquad \Omega_t = 1.88 \text{ (ASD)}$$

(b) For tensile rupture in the net section

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

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

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, which for cold-formed HSS, may be replaced with $F_{y,avg}$ determined in accordance with Section B4.3, ksi (MPa)

F_u = specified minimum tensile strength, ksi (MPa)

User Note: Due to the high ductility and strain hardening in some stainless steels, the limit state of tensile rupture in the net section may be associated with large deformation. If it is necessary to limit the deformation, the stress in the net section may be limited to $f_{max} < F_u$, based on the maximum member elongation permitted, Δ, as:

$$f_{max} = F_y + \left(\frac{F_u - F_y}{\varepsilon_u - F_y/E}\right)\left(\frac{\Delta}{L} - \frac{F_y}{E}\right)$$

where

E = modulus of elasticity of stainless steel, ksi (MPa)

= 28,000 ksi (193 000 MPa) for austenitic, 29,000 ksi (200 000 MPa) for duplex, 28,500 ksi (197 000 MPa) for precipitation hardening stainless steel, and 25,000 ksi (172 000 MPa) for ASTM A276/A276M (condition B) cold-finished austenitic stainless steel bar

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

ε = strain at the ultimate stress, which may be approximated in accordance with Equation A-7-3 if not known

Alternatively, a more accurate determination of the maximum stress in the net section can be obtained by rearranging Equation A-7-1b and replacing ε with Δ/L. There is no reason to consider a strength lower than $F_y A_e$.

User Note: Appendix 2 gives an alternative method for determining the tensile strength of structural members made of austenitic or duplex stainless steel that accounts for the beneficial effect of strain hardening.

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 austenitic and duplex stainless steel tension members shall be determined in accordance with the provisions of Section B4.4.

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, 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, 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.

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

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 austenitic and duplex stainless steel pin-connected members, shall be the lower value determined according to the limit states of tensile yielding, tensile rupture, shear rupture, and bearing.

(a) For tensile yielding on the gross section, Section D2(a) applies.

(b) For tensile rupture on the net effective area, Section D2(b) applies, with the effective net area, A_e, taken as

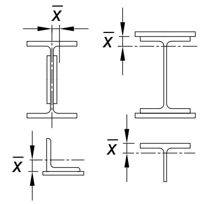
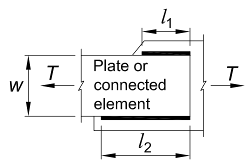
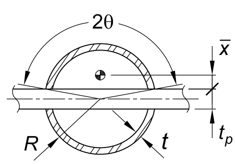
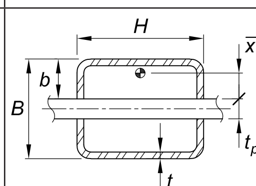
$$A_e = 2tb_e \tag{D5-1}$$

(c) For shear rupture on the effective area

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

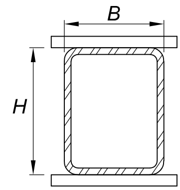
$$\phi_{sf} = 0.75 \text{ (LRFD)} \qquad \Omega_{sf} = 2.00 \text{ (ASD)}$$

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 and S shapes. (For angles, Case 8 is permitted to be used.)	$U = 1 - \dfrac{\bar{x}}{l}$	
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.	$\bar{x} = \dfrac{R\sin\theta}{\theta} - \dfrac{1}{2}t_p$ where θ is in rad $U = \left[1 + \left(\dfrac{\bar{x}}{l}\right)^{3.2}\right]^{-10}$	
		$\bar{x} = b - \dfrac{2b^2 + tH - 2t^2}{2H + 4b - 4t}$ $U = 1 - \dfrac{\bar{x}}{l}$	

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

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

Case	Description of Element		Shear Lag Factor, U	Examples
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- or S-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	$b_f \geq \frac{2}{3}d,\ U = 0.90$ $b_f < \frac{2}{3}d,\ U = 0.85$	–
		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); b = distance from face of plate to edge of rectangular HSS member, measured 90° to the plane of the connection, in. (mm); b_f = width of flange, 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); t = design wall thickness of HSS member, in. (mm); t_p = design thickness of plate, in. (mm); w = width of plate, in. (mm); $\bar{x}$ = eccentricity of connection, in. (mm).

where

A_{sf} = area on the shear failure path, in.^{2} (mm^2)
= $2t\left(a + d_{pin}/2\right)$

a = shortest distance from the edge of the pin hole to the edge of the member measured parallel to the direction of the force, in. (mm)

b_e = distance from the edge of the hole to the edge of the part, measured in the direction normal to the applied force, in. (mm), but not greater than $2t + 0.63$, in. (= $2t + 16$, mm)

d_{pin} = diameter of pin, in. (mm)

t = design thickness of plate, as defined in Section B4.2, in. (mm)

(d) For bearing on the projected area of the pin, Section J7 applies.

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 1⁄16 in. (1.5 mm) greater than the diameter of the pin.

(c) The width of the plate at the pin hole shall not be less than twice the diameter of the pin hole.

(d) The minimum extension, a, beyond the bearing end of the pin hole, parallel to the axis of the member, shall not be less than $1.33b_e$.

(e) The end beyond the pin hole may be chamfered, filleted, or trimmed, 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.

(f) The design thickness of the plate shall be enough to satisfy the bearing strength limits of the pin material given in Section J8.

User Note: The provisions for designing the pin are included in Section J8.

CHAPTER E

DESIGN OF MEMBERS FOR COMPRESSION

This chapter addresses austenitic and duplex stainless steel members subject to axial compression.

The chapter is organized as follows:

- E1. General Provisions
- E2. Effective Length
- E3. Flexural Buckling of Members without Slender Elements
- E4. Torsional and Flexural-Torsional Buckling of Single Equal-Leg Angles and Members without Slender Elements
- E5. Single Equal-Leg-Angle Compression Members without Slender Elements
- E6. Built-Up Members
- E7. Members with Slender Elements

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

User Note: The design of unequal-leg angles is outside the scope of this chapter due to insufficient research and test data to substantiate torsional and flexural-torsional buckling of these members.

E1. GENERAL PROVISIONS

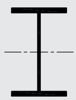
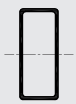
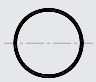
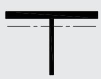
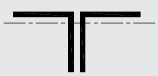
The design compressive strength, $\phi_c P_n$, and the allowable compressive strength, P_n/Ω_c, are determined as follows.

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.

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

User Note: For columns with short unbraced lengths, Appendix 2 gives an alternative method for determining the compressive strength of austenitic or duplex stainless steel I-shaped members, channels, angles, tees, HSS, and box-section members that accounts for the beneficial effect of strain hardening.

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	FB FTB	N/A	N/A
	E3	FB	N/A	N/A

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,

where

K = effective length factor

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

= KL

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, can be determined through methods other than those using the effective length factor, K.

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 design of wide-flange and similarly shaped columns.

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

$$P_n = F_{cr} A_g \tag{E3-1}$$

The critical stress, F_{cr}, is determined as follows:

(a) When $\dfrac{L_c}{r} \le \beta_0 \sqrt{\dfrac{E}{F_y}}$ $\left[\text{or } \dfrac{F_y}{F_e} \le \left(\dfrac{\beta_0}{\pi}\right)^2\right]$

$$F_{cr} = F_y \tag{E3-2}$$

(b) When $\beta_0 \sqrt{\dfrac{E}{F_y}} < \dfrac{L_c}{r} \le 5.62 \sqrt{\dfrac{E}{F_y}}$ $\left[\text{or } \left(\dfrac{\beta_0}{\pi}\right)^2 < \dfrac{F_y}{F_e} \le 3.20\right]$

$$F_{cr} = 1.2\left[\beta_1^{\left(\frac{F_y}{F_e}\right)^{\alpha}}\right] F_y \tag{E3-3}$$

(c) When $\dfrac{L_c}{r} > 5.62 \sqrt{\dfrac{E}{F_y}}$ $\left(\text{or } \dfrac{F_y}{F_e} > 3.20\right)$

$$F_{cr} = \beta_2 F_e \tag{E3-4}$$

where

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

E = modulus of elasticity of stainless steel, ksi (MPa)
= 28,000 ksi (193 000 MPa) for austenitic, and 29,000 ksi (200 000 MPa) for duplex stainless steel

F_e = elastic buckling stress determined according to Equation E3-5 or through an elastic buckling analysis, as applicable, ksi (MPa)

$$= \frac{\pi^2 E}{\left(\dfrac{L_c}{r}\right)^2} \tag{E3-5}$$

F_y = specified minimum yield stress, which for cold-formed HSS, may be replaced by $F_{y,avg}$ determined in accordance with Section B4.3, ksi (MPa)

r = radius of gyration, in. (mm)

α, β_0, β_1, β_2 = flexural buckling coefficients determined from Table E3.1

User Note: The two inequalities for calculating the limits of applicability of Sections E3(a) to E3(c), 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 EQUAL-LEG ANGLES AND MEMBERS WITHOUT SLENDER ELEMENTS

This section applies to channels, tees, and single equal-leg angles; certain doubly symmetric members, such as built-up members; and doubly symmetric members when the torsional unbraced length exceeds the lateral unbraced length, all without slender elements.

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

$$P_n = F_{cr} A_g \tag{E4-1}$$

The critical stress, F_{cr}, shall be determined according to Equations E3-2 through E3-4 and curve A in Table E3.1, and, 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

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

TABLE E3.1
Flexural Buckling Coefficients

Member Type	Alloy Family	Curve	α	β_0	β_1	β_2
Rolled or built-up I-shaped sections buckling about the minor axis, and other sections not specified in this table	Austenitic and duplex	A	0.56	0.759	0.409	0.690
Rolled or built-up I-shaped sections buckling about the major axis, welded box sections, and round HSS	Austenitic and duplex	B	0.58	0.891	0.455	0.820
Rectangular HSS	Austenitic and duplex	C	0.69	1.195	0.501	0.820

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}.

where

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

$$F_{ey} = \frac{\pi^2 E}{\left(\dfrac{L_{cy}}{r_y}\right)^2} \tag{E4-4}$$

$$F_{ez} = \left(\frac{\pi^2 E C_w}{L_{cz}^2} + GJ\right)\frac{1}{A_g \bar{r}_o^2} \tag{E4-5}$$

G = shear modulus of elasticity of stainless steel, ksi (MPa)
= 10,800 ksi (74 500 MPa) for austenitic, and 11,200 ksi (77 200 MPa) for duplex stainless steel

H = flexural constant

$$= 1 - \frac{x_o^2 + y_o^2}{\bar{r}_o^2} \tag{E4-6}$$

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

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

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

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

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

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

L_y, L_z = laterally unbraced length of the member for the *y*- and longitudinal axes, in. (mm)

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

$$\bar{r}_o^2 = x_o^2 + y_o^2 + \frac{I_x + I_y}{A_g} \tag{E4-7}$$

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}.

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

E5. SINGLE EQUAL-LEG ANGLE COMPRESSION MEMBERS WITHOUT SLENDER ELEMENTS

The nominal compressive strength, P_n, of single equal-leg angle members without slender elements shall be the lowest value based on the limit states of flexural buckling in accordance with Section E3 or flexural-torsional buckling in accordance with Section E4.

The effects of eccentricity on single equal-leg 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.

Single equal-leg angle members that do not meet these requirements or the requirements described in Section E5(a) or (b) are outside the scope of this Specification.

(a) For equal-leg 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) When $\frac{L}{r_a} \leq 80$

$$\frac{L_c}{r} = 72 + 0.75\frac{L}{r_a} \tag{E5-1}$$

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

$$\frac{L_c}{r} = 32 + 1.25\frac{L}{r_a} \tag{E5-2}$$

(b) For equal-leg 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) When $\dfrac{L}{r_a} \le 75$

$$\frac{L_c}{r} = 60 + 0.8\frac{L}{r_a} \tag{E5-3}$$

(2) When $\dfrac{L}{r_a} > 75$

$$\frac{L_c}{r} = 45 + \frac{L}{r_a} \tag{E5-4}$$

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)
= KL

r_a = radius of gyration about the geometric axis, in. (mm)

E6. BUILT-UP MEMBERS

1. Compressive Strength

This section applies to singly and doubly symmetric built-up members composed of two shapes interconnected by bolts or welds. The end connection shall be welded or connected by means of pretensioned bolts.

> **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 singly and doubly symmetric 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 $\left(L_c/r\right)_m$, determined as follows:

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

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

(b) For intermediate connectors that are welded or are connected by means of pretensioned bolts with faying surfaces as specified in Table J3.4

(1) When $\frac{a}{r_i} \le 40$

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

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

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

where

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

$\left(\frac{L_c}{r}\right)_o$ = 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. Dimensional Requirements

Singly and doubly symmetric 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 least 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 1.5 times the maximum width of the member.

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, excluding single angles.

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:

$$P_n = F_{cr} A_e \tag{E7-1}$$

where

A_e = summation of the effective areas of the cross section based on reduced effective widths, b_e, d_e, or h_e, in.^{2} (mm^2)

F_{cr} = critical stress determined in accordance with Section E3 or E4, ksi (MPa)

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 $(b - b_e)t$.

User Note: Alternative design rules for slender-element compression members with open cross sections, cold-formed to shape from annealed and cold-rolled stainless steel sheet, strip, or plate of less than 1 in. (25 mm) are given in the AISC *Specification for the Design of Cold-Formed Stainless Steel Structural Members* (ASCE/SEI 8-21).

1. Slender Element Members Excluding Round HSS

(a) The effective width, b_e, (for tee stems, this is d_e; for webs, this is h_e) for slender elements is determined as follows:

(1) When $\lambda \le \lambda_r \sqrt{\dfrac{F_y}{F_{cr}}}$

$$b_e = b \tag{E7-2}$$

(2) When $\lambda > \lambda_r \sqrt{\dfrac{F_y}{F_{cr}}}$

$$b_e = 0.772b\left(1 - 0.10\sqrt{\frac{F_{el}}{F_{cr}}}\right)\sqrt{\frac{F_{el}}{F_{cr}}} \tag{E7-3}$$

where

F_{el} = elastic local buckling stress of the full cross section, or conservatively determined according to Equation E7-4 for each individual element of the cross section, ksi (MPa)

$$= k\frac{\pi^2 E}{12\left(1-\nu^2\right)}\left(\frac{1}{\lambda}\right)^2 \tag{E7-4}$$

F_y = specified minimum yield stress, which for cold-formed square or rectangular HSS, may be replaced by $F_{y,avg}$ determined in accordance with Section B4.3, ksi (MPa)

b = width of the element (for tee stems this is d and for webs this is h, as defined in Section B4.1), in. (mm)

k = plate buckling coefficient
= 0.425 for unstiffened elements in compression
= 4.00 for stiffened elements in compression

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
ν = Poisson's ratio
= 0.3

User Note: The Commentary on Appendix 1, Section 1.3, gives analytical expressions for determining the elastic local buckling stress for the full cross section of I-shaped sections and square and rectangular HSS. Alternatively, the elastic buckling stress of the full cross section may be determined using numerical methods.

(b) Alternatively, for cold-formed square and rectangular HSS, the effective width, b_e, is permitted to be determined as follows:

(1) When $\lambda \le \lambda_r \sqrt{\dfrac{F_y}{F_{cr}}}$

$$b_e = b \tag{E7-5}$$

(2) When $\lambda > \lambda_r \sqrt{\dfrac{F_y}{F_{cr}}}$

$$b_e = b\left(1 - 0.22\sqrt{\frac{F_{el}}{F_{cr}}}\right)\sqrt{\frac{F_{el}}{F_{cr}}} \tag{E7-6}$$

If Equations E7-5 and E7-6 are used to calculate b_e, then the use of $F_{y,avg}$ is not permitted.

2. Round HSS

The effective area, A_e, is determined as follows:

(a) When $\lambda \le \lambda_r \dfrac{F_y}{F_{cr}}$

$$A_e = A_g \tag{E7-7}$$

(b) When $\lambda_r \dfrac{F_y}{F_{cr}} < \lambda < 2.8\lambda_r \dfrac{F_y}{F_{cr}}$

$$A_e = \left(\frac{0.6\lambda_r}{\lambda}\frac{F_y}{F_{cr}} + \frac{2}{5}\right)A_g \tag{E7-8}$$

where
D = outside diameter of round HSS, in. (mm)
F_y = specified minimum yield stress, which for cold-formed round HSS, may be replaced by $F_{y,avg}$ determined in accordance with Section B4.3, ksi (MPa)
t = design wall thickness, as defined in Section B4.2, in. (mm)
λ = $\dfrac{D}{t}$
λ_r = limiting width-to-thickness ratio as defined in Table B4.1a

CHAPTER F

DESIGN OF MEMBERS FOR FLEXURE

This chapter applies to austenitic and duplex stainless steel 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 and Channels with Compact Webs and Noncompact or Slender Flanges Bent about Their Major Axis
F4. Doubly Symmetric I-Shaped Members with Noncompact Webs Bent about Their Major Axis
F5. Double 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. Solid Rectangular Shapes and Rounds
F10. Other Shapes
F11. Proportions of Beams and Girders

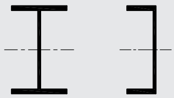
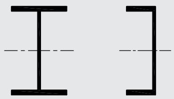
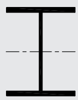
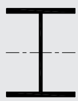
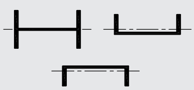
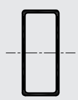
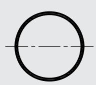
User Note: For cases not included in this chapter, the following provisions apply:

- Chapter G — Design provisions for shear
- Sections H1–H3 — Members subject to biaxial flexure or to combined flexure and axial force
- 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.

User Note: The design of single angles in flexure is outside the scope of this chapter due to insufficient research and test data to substantiate the design of these members.

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	NC	CFY, LTB, FLB
F5		C, NC, S	S	CFY, LTB, FLB
F6		C, NC, S	C, NC, S	Y, FLB, WLB
F7		C, NC, S	C, NC, S	Y, FLB, WLB
F8		N/A	N/A	Y, LB
F9		N/A	N/A	Y, LTB
F10	Other 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, 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

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

and the nominal flexural strength, M_n, shall be determined according to Sections F2 through F11.

User Note: Alternative design rules for slender-element flexural members with open cross sections, cold-formed to shape from annealed and cold-rolled stainless steel sheet, strip, or plate of less than 1 in. (25 mm) are given in the ASCE *Specification for the Design of Cold-Formed Stainless Steel Structural Members* (ASCE/SEI 8-21).

(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:

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

where

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

M_A = absolute value of moment at one-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 on the AISC *Specification for Structural Steel Buildings*, hereafter referred as the 2016 AISC *Specification*. The Commentary on the 2016 AISC *Specification* 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.

User Note: When applicable, Appendix 2 gives an alternative method for determining the flexural strength of austenitic and duplex stainless steel laterally restrained I-shaped members, channels, angles, tees, HSS, and box-section members that accounts for the beneficial effect of strain hardening.

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.

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, ksi (MPa)

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

2. Lateral-Torsional Buckling

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

(b) When $L_p \le L_b \le L_y$

$$M_n = C_b \left[M_p - \left(M_p - F_y S_x \right) \left(\frac{L_b - L_p}{L_y - L_p} \right) \right] \le M_p \tag{F2-2}$$

(c) When $L_y \le L_b \le L_r$

$$M_n = C_b \left[M_y - \left(M_y - 0.30 F_y S_x \right) \left(\frac{L_b - L_y}{L_r - L_y} \right)^{\alpha_{LT}} \right] \le M_p \tag{F2-3}$$

(d) When $L_b > L_r$

$$M_n = \beta_{LT} F_{cr} S_x \le M_p \tag{F2-4}$$

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} = critical stress, ksi (MPa)

$$= \frac{C_b \pi^2 E}{\left(\dfrac{L_b}{r_{ts}}\right)^2} \sqrt{1 + 0.078 \frac{Jc}{S_x h_o} \left(\frac{L_b}{r_{ts}}\right)^2} \tag{F2-5}$$

E = modulus of elasticity of stainless steel, ksi (MPa)
= 28,000 ksi (193 000 MPa) for austenitic, and 29,000 ksi (200 000 MPa) for duplex stainless steel

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

M_y = yield moment, kip-in. (N-mm)
= $F_y S_x$

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

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

$$\alpha_{LT} = 0.60 - 0.40 \left(\frac{L_b - L_y}{L_r - L_y}\right) \tag{F2-6}$$

β_{LT} = elastic lateral-torsional buckling reduction coefficient determined from Table F2.1

User Note: The square root term in Equation F2-5 may be conservatively taken equal to 1.0.

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

$$L_p = \beta_{p,LT} r_{ts} \sqrt{\frac{E}{F_y}} \tag{F2-7}$$

L_y, the laterally unbraced length required to achieve the yield moment, in. (mm), is:

$$L_y = 1.95 \beta_{y,LT} r_{ts} \frac{\beta_{LT} E}{F_y} \sqrt{\frac{Jc}{S_x h_o} + \sqrt{\left(\frac{Jc}{S_x h_o}\right)^2 + 6.76 \left(\frac{F_y}{\beta_{LT} E}\right)^2}} \tag{F2-8}$$

L_r, the limiting unbraced length for the limit state of inelastic lateral-torsional buckling, in. (mm), is:

$$L_r = 1.95 r_{ts} \frac{\beta_{LT} E}{0.30 F_y} \sqrt{\frac{Jc}{S_x h_o} + \sqrt{\left(\frac{Jc}{S_x h_o}\right)^2 + 6.76 \left(\frac{0.30 F_y}{\beta_{LT} E}\right)^2}} \tag{F2-9}$$

where

$\beta_{p,LT}$, $\beta_{y,LT}$ = lateral-torsional buckling coefficient determined from Table F2.1

$$r_{ts}^2 = \frac{\sqrt{I_y C_w}}{S_x} \tag{F2-10}$$

and the coefficient c is determined as follows:

TABLE F2.1
Lateral-Torsional Buckling Coefficients

Alloy Family	β_{LT}	$\beta_{p,LT}$	$\beta_{y,LT}$
Austenitic	0.82	0.90	0.40
Duplex	0.86	1.10	0.50

(1) For doubly symmetric I-shapes

$$c = 1 \tag{F2-11a}$$

(2) For channels

$$c = \frac{h_o}{2}\sqrt{\frac{I_y}{C_w}} \tag{F2-11b}$$

where

I_y = moment of inertia about the y-axis, in.^{4} (mm^4)

User Note: For doubly symmetric I-shapes with rectangular flanges, $C_w = \dfrac{I_y h_o^2}{4}$, and thus, Equation F2-10 becomes

$$r_{ts}^2 = \frac{I_y h_o}{2S_x}$$

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

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

F3. DOUBLY SYMMETRIC I-SHAPED MEMBERS AND CHANNELS WITH COMPACT WEBS AND NONCOMPACT OR SLENDER FLANGES 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 noncompact or slender flanges as defined in Section B4.1 for flexure.

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

$$M_n = M_p - \left(M_p - F_y S_x\right)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \tag{F3-1}$$

(b) For sections with slender flanges

$$M_n = F_y S_{xe} \tag{F3-2}$$

where

S_{xe} = effective section modulus, in.^{3} (mm^3), referred to the extreme compressive fibers with respect to the neutral axis of the effective cross section determined with the effective width, b_e, of the compression flange taken as:

$$b_e = 0.772b\left(1 - 0.10\sqrt{\frac{F_{el}}{F_y}}\right)\sqrt{\frac{F_{el}}{F_y}} \le b \tag{F3-3}$$

F_{el} = elastic local buckling stress of the full cross section, or conservatively determined for the compression flange according to Equation F3-4, ksi (MPa)

$$= k\frac{\pi^2 E}{12\left(1-\nu^2\right)}\left(\frac{1}{\lambda}\right)^2 \tag{F3-4}$$

b = width of element; 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)

k = plate buckling coefficient
= 0.425 for the compression flange of I-shaped members and channels

λ = $\dfrac{b}{t_f}$

t_f = design thickness of the flange, as defined in Section B4.2, in. (mm)

λ_{pf} = λ_p, the limiting width-to-thickness ratio for a compact flange, defined in Table B4.1b

λ_{rf} = λ_r, the limiting width-to-thickness ratio for a noncompact flange, defined in Table B4.1b

User Note: The Commentary on Appendix 1, Section 1.3, gives analytical expressions for determining the elastic local buckling stress for the full cross section of I-shaped sections. Alternatively, the elastic buckling stress of the full cross section may be determined using numerical methods.

F4. DOUBLY SYMMETRIC I-SHAPED MEMBERS WITH NONCOMPACT WEBS BENT ABOUT THEIR MAJOR AXIS

This section applies to doubly symmetric I-shaped members bent about their major axis with noncompact webs, 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, and compression flange local buckling.

1. Compression Flange Yielding

$$M_n = R_p M_y \tag{F4-1}$$

where

M_y = yield moment, kip-in. (N-mm)
$= F_y S_x$

R_p = web plastification factor, determined in accordance with Section F4.2

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

2. Lateral-Torsional Buckling

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

(b) When $L_p < L_b \le L_y$

$$M_n = C_b\left[M_p - \left(M_p - F_y S_x\right)\left(\frac{L_b - L_p}{L_y - L_p}\right)\right] \le R_p M_y \tag{F4-2}$$

(c) When $L_y < L_b \le L_r$

$$M_n = C_b\left[M_y - \left(M_y - 0.30 F_y S_x\right)\left(\frac{L_b - L_y}{L_r - L_y}\right)^{\alpha_{LT}}\right] \le R_p M_y \tag{F4-3}$$

(d) When $L_b > L_r$

$$M_n = \beta_{LT} F_{cr} S_x \le R_p M_y \tag{F4-4}$$

where

α_{LT} = lateral-torsional buckling coefficient determined by Equation F2-6

β_{LT} = elastic lateral-torsional buckling reduction coefficient determined from Table F2.1

F_{cr} = critical stress, determined by Equation F2-5, ksi (MPa)

L_p = limiting laterally unbraced length for the limit state of yielding, determined by Equation F2-7, in. (mm)

L_r = limiting laterally unbraced length for the limit state of inelastic lateral-torsional buckling, determined by Equation F2-9, in. (mm)

L_y = laterally unbraced length required to achieve the yield moment, determined by Equation F2-8, in. (mm)

The web plastification factor, R_p, is determined as follows:

(1) When $I_{yf}/I_y > 0.23$

$$R_p = \left[\frac{M_p}{M_y} - \left(\frac{M_p}{M_y} - 1\right)\left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right)\right] \le \frac{M_p}{M_y} \tag{F4-5a}$$

(2) When $I_{yf}/I_y \le 0.23$

$$R_p = 1.0 \tag{F4-5b}$$

where

I_{yf} = moment of inertia of the flange about the y-axis, in.^{4} (mm^4)

M_p = $F_y Z_x$

h = depth of web, as defined in Section B4.1b, in. (mm)

λ = $\dfrac{h}{t_w}$

λ_{pw} = λ_p, the limiting width-to-thickness ratio for a compact web, given in Table B4.1b

λ_{rw} = λ_r, the limiting width-to-thickness ratio for a noncompact web, given in Table B4.1b

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

$$M_n = R_p M_y - \left(R_p M_y - F_y S_x\right)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \tag{F4-6}$$

(c) For sections with slender flanges

$$M_n = F_y S_{xe} \tag{F4-7}$$

where

S_{xe} = effective section modulus referred to the extreme compressive fibers with respect to the neutral axis of the effective cross section determined with the effective width, b_e, of the compression flange given by Equation F3-3, and based on the elastic Equation F3-4, in.^{3} (mm^3)

R_p = web plastification factor, determined by Equation F4-5a or F4-5b

λ = $\dfrac{b_f}{2t_f}$

λ_{pf} = λ_p, the limiting width-to-thickness ratio for a compact flange, defined in Table B4.1b

λ_{rf} = λ_r, the limiting width-to-thickness ratio for a noncompact flange, defined in Table B4.1b

F5. DOUBLY SYMMETRIC I-SHAPED MEMBERS WITH SLENDER WEBS BENT ABOUT THEIR MAJOR AXIS

This section applies to doubly symmetric I-shaped members with slender webs 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, and compression flange local buckling.

1. Compression Flange Yielding

$$M_n = R_{pg} F_y S_x \tag{F5-1}$$

2. Lateral-Torsional Buckling

(a) When $L_b \le L_y$, the limit state of lateral-torsional buckling does not apply.

(b) When $L_y < L_b \le L_r$

$$M_n = C_b R_{pg} \left[M_y - \left(M_y - 0.30 F_y S_x \right) \left(\frac{L_b - L_y}{L_r - L_y} \right)^{\alpha_{LT}} \right] \le R_{pg} F_y S_x \tag{F5-2}$$

(c) When $L_b > L_r$

$$M_n = \beta_{LT} R_{pg} F_{cr} S_x \le R_{pg} F_y S_x \tag{F5-3}$$

where

α_{LT} = lateral-torsional buckling coefficient determined by Equation F2-6

β_{LT} = elastic lateral-torsional buckling reduction coefficient determined from Table F2.1

F_{cr} = critical stress, determined by Equation F2-5

L_y = laterally unbraced length required to achieve the yield moment, determined by Equation F2-8

L_r = limiting laterally unbraced length for the limit state of inelastic lateral-torsional buckling, determined by Equation F2-9

R_{pg} = bending strength reduction factor

$$= 1 - \frac{a_w}{6.75 + 2.12 a_w} \left(\frac{\lambda}{\lambda_{rw}} - 1.0 \right) \le 1.0 \tag{F5-4}$$

where

$$a_w = \frac{h t_w}{b_f t_f} \le 10.0 \tag{F5-5}$$

b_f = width of flange, in. (mm)

h = depth of web, as defined in Section B4.1b, in. (mm)

t_f = design thickness of flange, as defined in Section B4.2, in. (mm)

t_w = design thickness of web, as defined in Section B4.2, in. (mm)

$\lambda = \dfrac{h}{t_w}$

λ_{rw} = λ_r, the limiting width-to-thickness ratio for a noncompact web, defined in Table B4.1b

3. Compression Flange Local Buckling

(a) For sections with compact or noncompact flanges, the limit state of compression flange local buckling does not apply.

(b) For sections with slender flanges

$$M_n = R_{pg} F_y S_{xe} \tag{F5-6}$$

where

R_{pg} = bending strength reduction factor, determined by Equation F5-4

S_{xe} = effective section modulus referred to the extreme compressive fibers with respect to the neutral axis of the effective cross section determined with the effective width, b_e, of the compression flange given by Equation F3-3, and based on the elastic local buckling stress of the full cross section, or conservatively determined for the compression flange according to Equation F3-4, in.^{3} (mm^3)

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), flange local buckling, and web local buckling.

1. Yielding

$$M_n = M_p = F_y Z_y \tag{F6-1}$$

where

Z_y = plastic section modulus taken about the y-axis, in.^{3} (mm^3)

2. Flange Local Buckling

The limit state of flange local buckling is only applicable to I-shaped members, and channels bent with the flange tips in compression.

(a) For sections with compact flanges, the limit state of flange local buckling does not apply.

(b) For sections with noncompact flanges

$$M_n = M_p - \left(M_p - F_y S_{yc}\right)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \tag{F6-2}$$

(c) For sections with slender flanges

$$M_n = F_y S_{ye} \tag{F6-3}$$

where

S_{yc} = elastic section modulus referred to the extreme compressive fibers, in.^{3} (mm^3)

S_{ye} = effective section modulus refered to the extreme compressive fibers with respect to the neutral axis of the effective cross section determined with the effective width, b_e, of the compression flange given by Equation F3-3, in.^{3} (mm^3)

b = width of element; 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 = design thickness of the flange, as defined in Section B4.2, in. (mm)

$\lambda = \dfrac{b}{t_f}$

$\lambda_{pf} = \lambda_p$, the limiting width-to-thickness ratio for a compact flange, defined in Table B4.1b

$\lambda_{rf} = \lambda_r$, the limiting width-to-thickness ratio for a noncompact flange, defined in Table B4.1b

3. Web Local Buckling

The limit state of web local buckling is only applicable to channels bent with the flange tips in tension.

(a) For channels with compact webs, the limit state of web local buckling does not apply.

(b) For channels with noncompact webs

$$M_n = M_p - \left(M_p - F_y S_{yt}\right)\left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right) \tag{F6-4}$$

(c) For channels with slender webs when $\lambda_{rw} < \lambda \leq 1.5\lambda_{rw}$

$$M_n = F_y S_{yt} \tag{F6-5}$$

where

S_{yt} = elastic section modulus referred to the extreme tensile fibers, in.^{3} (mm^3)

$\lambda = \dfrac{h}{t_w}$

h = depth of web, as defined in Section B4.1b, in. (mm)

t_w = design thickness of the web, as defined in Section B4.2, in. (mm)

$\lambda_{pw} = \lambda_p$, the limiting width-to-thickness ratio for a compact web, defined in Table B4.1b

$\lambda_{rw} = \lambda_r$, the limiting width-to-thickness ratio for a noncompact web, defined in Table B4.1b

F7. SQUARE AND RECTANGULAR HSS AND BOX SECTIONS

This section applies to square and rectangular HSS and doubly symmetric box sections with $h/b \leq 3$ 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, and web local buckling under pure flexure.

User Note: In most practical cases, rectangular HSS and box sections with $h/b \le 3$ will not be susceptible to lateral-torsional buckling. For longer lengths, beam deflection is likely to be critical.

1. Yielding

$$M_n = M_p = F_y Z \tag{F7-1}$$

where

F_y = specified minimum yield stress, which for cold-formed square or rectangular HSS, may be replaced by $F_{y,avg}$ determined in accordance with Section B4.3, ksi (MPa)

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

$$M_n = M_p - \left(M_p - F_y S\right)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \le M_p \tag{F7-2}$$

(c) For sections with slender flanges

$$M_n = F_y S_e \tag{F7-3}$$

where

S = elastic section modulus about the axis of bending, in.^{3} (mm^3)

S_e = effective section modulus referred to the extreme compressive fiber with respect to the neutral axis of the effective cross section determined with the effective width, b_e, of the compression flange given by Equation F3-3, and based on the elastic local buckling stress of the full cross section, or conservatively determined for the compression flange according to Equation F3-4, in.^{3} (mm^3)

$\lambda = \dfrac{b}{t_f}$

b = width of compression flange as defined in Section B4.1b, in. (mm)

t_f = design thickness of the flange, as defined in Section B4.2, in. (mm)

λ_{pf} = λ_p, the limiting width-to-thickness ratio for a compact flange, defined in Table B4.1b

λ_{rf} = λ_r, the limiting width-to-thickness ratio for a noncompact flange, defined in Table B4.1b

User Note: The Commentary on Appendix 1, Section 1.3, gives analytical expressions for determining the elastic local buckling stress for the full cross section of square and rectangular HSS and box sections. Alternatively, the elastic buckling stress of the full cross section may be determined using numerical methods.

Alternatively, for cold-formed square and rectangular HSS, it is permitted to calculate the effective section modulus, S_e, based on the effective width of the compression flange, b_e, given by Equation F7-4. However, if Equation F7-4 is used, then the use of $F_{y,avg}$ when calculating b_e or M_n is not permitted.

$$b_e = b\left(1-0.22\sqrt{\frac{F_{el}}{F_y}}\right)\sqrt{\frac{F_{el}}{F_y}} \le b \tag{F7-4}$$

3. Web Local Buckling

(a) For compact sections, the limit state of web local buckling does not apply.

(b) For sections with noncompact webs

$$M_n = M_p - \left(M_p - F_y S\right)\left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right) \le M_p \tag{F7-5}$$

(c) For sections with slender webs

(1) Compression flange yielding

$$M_n = R_{pg} F_y S \tag{F7-6}$$

(2) Compression flange local buckling

$$M_n = R_{pg} F_y S_e \tag{F7-7}$$

where

R_{pg} = bending strength reduction factor, determined by Equation F5-4 with $a_w = 2ht_w/\left(bt_f\right)$

S_e = effective section modulus, as defined in Section F7.2

h = depth of web, as defined in Section B4.1b, in. (mm)

t_w = design thickness of the web, as defined in Section B4.2, in. (mm)

$\lambda = \dfrac{h}{t_w}$

λ_{pw} = λ_p, the limiting width-to-thickness ratio for a compact web, defined in Table B4.1b

λ_{rw} = λ_r, the limiting width-to-thickness ratio for a noncompact web, defined in Table B4.1b

F8. ROUND HSS

This section applies to compact and noncompact round HSS, as defined in Table B4.1b.

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

$$M_n = M_p = F_y Z \tag{F8-1}$$

2. Local Buckling

(a) For compact sections, the limit state of flange local buckling does not apply.

(b) For noncompact sections

$$M_n = \left(\frac{0.068\lambda_r}{\lambda} + 1\right) F_y S \tag{F8-2}$$

where

$\lambda = \dfrac{D}{t}$

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

t = design wall thickness of HSS member, as defined in Section B4.2, in. (mm)

λ_r = limiting width-to-thickness ratio as defined in Table B4.1b

F9. SOLID RECTANGULAR SHAPES AND ROUNDS

This section applies to solid rectangular shapes 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 solid rectangular shapes and rounds

$$M_n = M_p = F_y Z \tag{F9-1}$$

2. Lateral-Torsional Buckling

(a) For solid rectangular shapes with $\dfrac{L_b d}{t^2} \le \dfrac{0.306E}{F_y}$ bent about their major axis, solid rectangular shapes bent about their minor axis, and rounds, the limit state of lateral-torsional buckling does not apply.

(b) For solid rectangular shapes with $\dfrac{0.306E}{F_y} < \dfrac{L_b d}{t^2} \le \dfrac{2.00E}{F_y}$ bent about their major axis

$$M_n = C_b \left[1.61 - 0.36\left(\frac{L_b d}{t^2}\right)\frac{F_y}{E}\right] M_y \le M_p \tag{F9-2}$$

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)

d = depth of solid rectangular shape, in. (mm)

t = width of rectangular bar parallel to axis of bending, in. (mm)

(c) For solid rectangular shapes with $\dfrac{L_b d}{t^2} > \dfrac{2.00E}{F_y}$ bent about their major axis

$$M_n = F_{cr} S_x \le M_p \tag{F9-3}$$

where

$$F_{cr} = \frac{1.78EC_b}{\frac{L_b d}{t^2}} \tag{F9-4}$$

F10. OTHER SHAPES

This section applies to other doubly and singly symmetric shapes, such as singly symmetric I-shaped members and tees, and all unsymmetrical shapes except single angles, all without slender elements.

User Note: Design rules for cold-formed stainless steel flexural members with slender elements are given in the ASCE *Specification for the Design of Cold-Formed Stainless Steel Structural Members* (ASCE/SEI 8-21).

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

1. Yielding

$$M_n = F_y S_{min} \tag{F10-1}$$

where

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

2. Lateral-Torsional Buckling

(a) When $L_b \le L_y$, the limit state of lateral-torsional buckling does not apply.

(b) When $L_y < L_b \le L_r$

$$M_n = C_b\left[F_y S_{min} - \left(F_y S_{min} - 0.30 F_y S_{min}\right)\left(\frac{L_b - L_y}{L_r - L_y}\right)^{\alpha_{LT}}\right] \le F_y S_{min} \tag{F10-2}$$

(c) When $L_b > L_r$

$$M_n = 0.82 F_{cr} S_{min} \le F_y S_{min} \tag{F10-3}$$

where

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

L_r = limiting laterally unbraced length for the limit state of inelastic lateral-torsional buckling, determined as the length at which $F_{cr} = 0.37F_y$, in. (mm)

L_y = laterally unbraced length required to achieve the yield moment, determined as 0.4 times the length at which $F_{cr} = 1.22F_y$, in. (mm)

α_{LT} = lateral-torsional buckling coefficient determined by Equation F2-6

F11. 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 with bolt 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 $F_u A_{fn} \geq F_y A_{fg}$, the limit state of tensile rupture does not apply.

(b) When $F_u A_{fn} < F_y A_{fg}$, the nominal flexural strength, M_n, at the location of the bolt holes in the tension flange shall not be taken greater than

$$M_n = \frac{F_u A_{fn}}{A_{fg}} S_x \tag{F11-1}$$

where

A_{fg} = gross area of tension flange, calculated in accordance with the provisions of Section B4.4a, in.^{2} (mm^2)

A_{fn} = net area of tension flange, calculated in accordance with the provisions of Section B4.4b, 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)

2. 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 AND TORSION

This chapter addresses austenitic and duplex stainless steel singly or doubly symmetric members subject to shear in the plane of the web or in the weak direction, austenitic and duplex stainless steel HSS and box sections subject to shear, and austenitic and duplex stainless steel doubly symmetric I-shaped members, channels, HSS, and box sections subject to torsion only.

The chapter is organized as follows:

G1. General Provisions
G2. I-Shaped Members and Channels Subject to Major-Axis Shear
G3. Rectangular HSS and Box Sections Subject to Shear
G4. Round HSS Subject to Shear
G5. Doubly Symmetric I-Shaped Members and Channels Subject to Minor-Axis Shear
G6. Other Singly or Doubly Symmetric Shapes Subject to Shear
G7. Beams and Girders with Web Openings Subject to Shear
G8. Round and Rectangular HSS and Box Sections Subject to Torsion
G9. Doubly Symmetric I-Shaped Members and Channels Subject to Torsion

User Note: For cases not included in this chapter, the following sections apply:
- J4.2 Shear strength of connecting elements
- J11.6 Web panel-zone shear

User Note: Single angles subject to shear are outside the scope of this chapter due to insufficient research and test data. Angles, tees, and singly symmetric I-shaped members subject to torsion are also not included.

G1. GENERAL PROVISIONS

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

(1) For the provisions in Sections G2 through G6

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

(2) The nominal shear strength, V_n, shall be determined according to Sections G2 through G6.

(b) The design torsional strength, $\phi_T T_n$, and the allowable torsional strength, T_n/Ω_T, shall be determined as follows:

(1) For the provisions in Sections G8 and G9

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

(2) The nominal torsional strength, T_n, shall be determined according to Sections G8 and G9.

G2. I-SHAPED MEMBERS AND CHANNELS SUBJECT TO MAJOR-AXIS SHEAR

Two methods of calculating shear strength are presented. The method in Section G2.1 utilizes post-buckling strength without considering tension field action. It is applicable for webs with and without stiffeners. The method presented in Section G2.2 is applicable to webs with stiffeners and utilizes post-buckling strength as modeled by tension field action.

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

A_w = area of web, the overall depth times the web design thickness, dt_w, in.^{2} (mm^2)

F_y = specified minimum yield stress, ksi (MPa)

(a) The web shear strength coefficient, C_{v1}, is determined as follows:

(1) When $\lambda \le 0.59\sqrt{k_v E/F_y}$

$$C_{v1} = 1.2 \tag{G2-2}$$

(2) When $\lambda > 0.59\sqrt{k_v E/F_y}$

$$C_{v1} = \frac{1.55\sqrt{k_v E/F_y}}{0.7\sqrt{k_v E/F_y} + \lambda} \tag{G2-3}$$

where

E = modulus of elasticity of stainless steel, ksi (MPa)

= 28,000 ksi (193 000 MPa) for austenitic, and 29,000 ksi (200 000 MPa) for duplex stainless steel

$\lambda = \dfrac{h}{t_w}$

h = for rolled I-shaped members, the clear distance between flanges less the fillet at each flange, in. (mm)

= for built-up welded sections or members, the clear distance between flanges, in. (mm)

= for built-up bolted members, the distance between fastener lines, in. (mm)

t_w = design thickness of web, as defined in Section B4.2, in. (mm)

(b) The web plate shear buckling coefficient, k_v, is determined as follows:

(1) For webs without transverse stiffeners

$$k_v = 5.34$$

(2) For webs with transverse stiffeners

$$k_v = 5 + \frac{5}{(a/h)^2} \qquad \text{(G2-4)}$$

$$= 5.34 \text{ when } a/h > 3.0$$

where

a = clear distance between transverse stiffeners, in. (mm)

2. Shear Strength of Interior Web Panels with $a/h \leq 3$ Considering Tension Field Action

The nominal shear strength, V_n, is determined as follows:

(a) When $\lambda \leq 0.65\sqrt{k_v E/F_y}$

$$V_n = 0.6 C_{v2} F_y A_w \qquad \text{(G2-5)}$$

(b) When $\lambda > 0.65\sqrt{k_v E/F_y}$

(1) When $2A_w/(A_{fc} + A_{ft}) \leq 2.5$, $h/b_{fc} \leq 6.0$, and $h/b_{ft} \leq 6.0$

$$V_n = 0.6 F_y A_w \left[C_{v2} + \frac{1 - C_{v2}}{1.15\sqrt{1 + (a/h)^2}} \right] \qquad \text{(G2-6)}$$

(2) Otherwise

$$V_n = 0.6 F_y A_w \left[C_{v2} + \frac{1 - C_{v2}}{1.15\left[a/h + \sqrt{1 + (a/h)^2} \right]} \right] \qquad \text{(G2-7)}$$

where

the web shear buckling coefficient, C_{v2}, is determined as follows:

(i) When $\lambda \leq 0.33\sqrt{k_v E/F_y}$

$$C_{v2} = 1.2 \qquad \text{(G2-8)}$$

(ii) When $0.33\sqrt{k_v E/F_y} < \lambda \leq 0.97\sqrt{k_v E/F_y}$

$$C_{v2} = 1.2 - 0.62\left(\frac{\lambda}{\sqrt{k_v E/F_y}} - 0.33 \right) \qquad \text{(G2-9)}$$

(iii) When $0.97\sqrt{k_v E/F_y} < \lambda \le 2.68\sqrt{k_v E/F_y}$

$$C_{v2} = \frac{5.02\sqrt{k_v E/F_y} - \lambda}{1.62\sqrt{k_v E/F_y} + 3.55\lambda} \tag{G2-10}$$

(iv) When $\lambda > 2.68\sqrt{k_v E/F_y}$

$$C_{v2} = \frac{1.51 k_v E}{\lambda^2 F_y} \tag{G2-11}$$

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
$\lambda = \dfrac{h}{t_w}$

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. Transverse Stiffeners

For transverse stiffeners, the following shall apply.

(a) Transverse stiffeners are not required where $\lambda \le 2.46\sqrt{E/F_y}$, 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 uplift tendency due to torsion in 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) $$(b/t)_{st} \le 0.41\sqrt{\frac{E}{F_{yst}}} \tag{G2-12}$$

(e) $$I_{st} \ge I_{st2} + \left(I_{st1} - I_{st2}\right)\rho_w \tag{G2-13}$$

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} = 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)

$$= \frac{h^4 \rho_{st}^{1.3}}{40}\left(\frac{F_{yw}}{E}\right)^{1.5} \tag{G2-14}$$

F_{yw} = specified minimum yield stress of the web material, ksi (MPa)

I_{st2} = 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)

$$= \left[\frac{2.5}{(a/h)^2} - 2\right] b_p t_w^3 \geq 0.5 b_p t_w^3 \tag{G2-15}$$

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 G2-15 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 G2-13 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. RECTANGULAR HSS AND BOX SECTIONS SUBJECT TO SHEAR

The nominal shear strength, V_n, of rectangular HSS and box section members shall be determined as:

$$V_n = 0.6F_y A_w C_{v2} \tag{G3-1}$$

where

A_w = $2ht$, in.^{2} (mm^2)

C_{v2} = web shear buckling coefficient, as defined in Section G2.2, with $\lambda = h/t$ 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)

G4. ROUND HSS SUBJECT TO SHEAR

The nominal shear strength, V_n, of round HSS, shall be determined as:

$$V_n = 0.6C_vF_yA_g/2 \tag{G4-1}$$

where

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

F_y = specified minimum yield stress, ksi (MPa)

The shear buckling strength coefficient, C_v, is determined as follows:

(a) When $\dfrac{L_v}{D} \leq 4.21\sqrt{\lambda}$

$$C_v = C_{vM} \tag{G4-2}$$

(b) When $\dfrac{L_v}{D} > 4.21\sqrt{\lambda}$

$$C_v = C_{vL} \tag{G4-3}$$

where

D = outside diameter, in. (mm)

L_v = distance from the point of maximum shear force to the point of zero shear force, in. (mm)

$\lambda = \dfrac{D}{t}$

(1) The shear buckling strength coefficient, C_{vM}, is determined as follows:

(i) When $\lambda \leq 0.32\left(\dfrac{E/F_y}{\sqrt{L_v/D}}\right)^{0.80}$

$$C_{vM} = 1.0 \tag{G4-4}$$

(ii) When $0.32\left(\dfrac{E/F_y}{\sqrt{L_v/D}}\right)^{0.80} < \lambda \leq 4.65\left(\dfrac{E/F_y}{\sqrt{L_v/D}}\right)^{0.80}$

$$C_{vM} = 1 - 0.61\left(0.47\sqrt{\frac{\sqrt{L_v/D}\,\lambda^{1.25}}{E/F_y}} - 0.23\right) \tag{G4-5}$$

(iii) When $\lambda > 4.65\left(\dfrac{E/F_y}{\sqrt{L_v/D}}\right)^{0.80}$

$$C_{vM} = \frac{2.67E}{\sqrt{L_v/D}\,\lambda^{1.25}F_y} \qquad \text{(G4-6)}$$

(2) The shear buckling strength coefficient, C_{vL}, is determined as follows:

(i) When $\lambda \le 0.24\left(E/F_y\right)^{0.67}$

$$C_{vL} = 1.0 \qquad \text{(G4-7)}$$

(ii) When $0.24\left(E/F_y\right)^{0.67} < \lambda \le 2.23\left(E/F_y\right)^{0.67}$

$$C_{vL} = 1 - 0.61\left(0.68\sqrt{\frac{\lambda^{1.50}}{E/F_y}} - 0.23\right) \qquad \text{(G4-8)}$$

(iii) When $\lambda > 2.23\left(E/F_y\right)^{0.67}$

$$C_{vL} = \frac{1.30E}{\lambda^{1.50}F_y} \qquad \text{(G4-9)}$$

G5. DOUBLY SYMMETRIC I-SHAPED MEMBERS AND CHANNELS SUBJECT TO MINOR-AXIS SHEAR

For doubly symmetric I-shaped members and channels loaded in the minor axis without torsion, the nominal shear strength, V_n, for each shear resisting element is:

$$V_n = 0.6F_y b_f t_f C_{v2} \qquad \text{(G5-1)}$$

where

C_{v2} = web shear buckling coefficient, as defined in Section G2.2 with $\lambda = b_f/2t_f$ for I-shaped members, or $\lambda = b_f/t_f$ for channels, and $k_v = 1.2$

b_f = width of flange, in. (mm)

t_f = design thickness of flange, as defined in Section B4.2, in. (mm)

G6. OTHER SINGLY OR DOUBLY SYMMETRIC SHAPES SUBJECT TO SHEAR

The nominal shear strength, V_n, of other singly or doubly symmetric shapes shall be determined as:

$$V_n = 0.6C_{v2}F_y A_w \qquad \text{(G6-1)}$$

where

A_w = area of element or elements resisting the shear force, taken as the sum of the overall depth times the element design thickness, in.^{2} (mm^2)

C_{v2} = web shear buckling coefficient, as defined in Section G2.2, with $\lambda = d/t$ and $k_v = 5$ for stiffened elements and $k_v = 1.2$ for unstiffened elements

d = width of element resisting the shear force, in. (mm)
= for built-up welded sections or members, the clear distance between flanges, h, in. (mm)
= for built-up bolted members, the distance between fastener lines, h, in. (mm)
= for tee sections, depth of tee stem, b, in. (mm)
t = element design thickness, as defined in Section B4.2, in. (mm)

G7. BEAMS AND GIRDERS WITH WEB OPENINGS SUBJECT TO SHEAR

The effect of all web openings on the shear strength of beams shall be determined. Reinforcement shall be provided when the required strength exceeds the available strength of the member at the opening.

G8. ROUND AND RECTANGULAR HSS AND BOX SECTIONS SUBJECT TO TORSION

The nominal torsional strength, T_n, of round and rectangular HSS, according to the limit states of torsional yielding and torsional buckling, shall be determined as follows:

$$T_n = 0.6CC_vF_y \qquad \text{(G8-1)}$$

where
C = HSS torsional constant, in.^{3} (mm^3)

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

For round HSS: $C = \dfrac{\pi(D^4 - D_i^4)}{32D/2} = \dfrac{\pi(D-t)^2 t}{2}$

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

1. Round HSS

The shear buckling strength coefficient, C_v, is determined as follows:

(a) When $\dfrac{L}{D} \le 4.21\sqrt{\lambda}$

$$C_v = C_{vM} \qquad \text{(G8-2)}$$

(b) When $\dfrac{L}{D} > 4.21\sqrt{\lambda}$

$$C_v = C_{vL} \qquad \text{(G8-3)}$$

where
D = outside diameter, in. (mm)
L = length of member, in. (mm)
t = design thickness, as defined in Section B4.2, in. (mm)
$\lambda = \dfrac{D}{t}$

(1) C_{vM} is determined as follows:

(i) When $\lambda \leq 0.26\left(\frac{E/F_y}{\sqrt{L/D}}\right)^{0.80}$

$$C_{vM} = 1.0 \tag{G8-4}$$

(ii) When $0.26\left(\frac{E/F_y}{\sqrt{L/D}}\right)^{0.80} < \lambda \leq 3.77\left(\frac{E/F_y}{\sqrt{L/D}}\right)^{0.80}$

$$C_{vM} = 1 - 0.61\left(0.54\sqrt{\frac{\sqrt{L/D}\,\lambda^{1.25}}{E/F_y}} - 0.23\right) \tag{G8-5}$$

(iii) When $\lambda > 3.77\left(\frac{E/F_y}{\sqrt{L/D}}\right)^{0.80}$

$$C_{vM} = \frac{2.05E}{\sqrt{L/D}\,\lambda^{1.25}F_y} \tag{G8-6}$$

(2) C_{vL} is determined as follows:

(i) When $\lambda \leq 0.20\left(E/F_y\right)^{0.67}$

$$C_{vL} = 1.0 \tag{G8-7}$$

(ii) When $0.20\left(E/F_y\right)^{0.67} < \lambda \leq 1.87\left(E/F_y\right)^{0.67}$

$$C_{vL} = 1 - 0.61\left(0.77\sqrt{\frac{\lambda^{1.50}}{E/F_y}} - 0.23\right) \tag{G8-8}$$

(iii) When $\lambda > 1.87\left(E/F_y\right)^{0.67}$

$$C_{vL} = \frac{E}{\lambda^{1.50}F_y} \tag{G8-9}$$

2. Rectangular HSS and Box Sections

The shear buckling strength coefficient, C_v, is determined as follows:

(a) When $\lambda \leq 0.74\sqrt{E/F_y}$

$$C_v = 1.2 \tag{G8-10}$$

(b) When $0.74\sqrt{E/F_y} < \lambda \le 2.17\sqrt{E/F_y}$

$$C_v = 1.2 - 0.28\left(\frac{\lambda}{\sqrt{E/F_y}} - 0.74\right) \tag{G8-11}$$

(c) When $2.17\sqrt{E/F_y} < \lambda \le 5.99\sqrt{E/F_y}$

$$C_v = \frac{11.23\sqrt{E/F_y} - \lambda}{3.62\sqrt{E/F_y} + 3.55\lambda} \tag{G8-12}$$

(d) When $\lambda > 5.99\sqrt{E/F_y}$

$$C_v = \frac{7.56E}{\lambda^2 F_y} \tag{G8-13}$$

where

$\lambda = \dfrac{h}{t}$

h = flat width of longer side, as defined in Section B4.1b(d), in. (mm)

t = design thickness, as defined in Section B4.2, corresponding to the longer side, in. (mm)

G9. DOUBLY SYMMETRIC I-SHAPED MEMBERS AND CHANNELS SUBJECT TO TORSION

The nominal torsional strength, T_n, of doubly symmetric I-shaped members and channels 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:

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

$$F_n = F_y \tag{G9-1}$$

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

$$F_n = 0.6F_y \tag{G9-2}$$

(c) For the limit state of buckling

$$F_n = F_{cr} \tag{G9-3}$$

where

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

CHAPTER H

DESIGN OF MEMBERS FOR COMBINED FORCES

This chapter addresses austenitic and duplex stainless steel doubly symmetric I-shaped members, channels, HSS, and box sections subject to axial force and flexure about one or both axes, with or without torsion.

The chapter is organized as follows:

User Note: When applicable, Appendix 2 gives an alternative method that accounts for the beneficial effect of strain hardening when determining the strength of laterally restrained I-shaped, HSS, and box-section members made of austenitic or duplex stainless steel subject to axial force and flexure about one or both axes.

User Note: The design of tees, single equal-leg angles, and double angles used in compression when other forces are present is outside the scope of this chapter.

H1. DOUBLY SYMMETRIC I-SHAPED MEMBERS, CHANNELS, HSS, AND BOX SECTIONS SUBJECT TO FLEXURE AND AXIAL FORCE

1. Doubly Symmetric I-Shaped Members, Channels, HSS, and Box Sections Subject to Flexure and Compression

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

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

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

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

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

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 Symmetric I-Shaped Members, Channels, HSS, and Box Sections Subject to Flexure and Tension

The interaction of flexure and tension in doubly symmetric members and channels 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

$$P_{ey} = \frac{\pi^2 EI_y}{L_b^2} \tag{H1-2}$$

α = 1.0 (LRFD)
= 1.6 (ASD)

E = modulus of elasticity of stainless steel, ksi (MPa)
= 28,000 ksi (193 000 MPa) for austenitic, and 29,000 ksi (200 000 MPa) for duplex stainless steel

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 to prevent twist of the cross section, in. (mm)

H2. DOUBLY SYMMETRIC I-SHAPED MEMBERS, CHANNELS, HSS, AND BOX SECTIONS SUBJECT TO COMBINED TORSION, FLEXURE, SHEAR, AND/OR AXIAL FORCE

1. HSS and Box Sections 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 and box sections 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

$$\left(\frac{P_r}{P_c}+\frac{M_r}{M_c}\right)+\left(\frac{V_r}{V_c}+\frac{T_r}{T_c}\right)^2 \leq 1.0 \qquad \text{(H2-1)}$$

where

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, $\phi_b M_n$ or M_n/Ω_b, determined in accordance with Chapter F, kip-in. (N-mm)

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)

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, $\phi_T T_n$ or T_n/Ω_T, determined in accordance with Section G8, 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, $\phi_v V_n$ or V_n/Ω_v, determined in accordance with Chapter G, kips (N)

2. Doubly Symmetric I-Shaped Members and Channels Subject to Combined Stress

The available strength of doubly symmetric I-shaped members and channels subject to torsion combined with shear, flexure, and/or axial force shall be the lowest value determined in accordance with Section G9 for the limit states of yielding under normal stress, shear yielding under shear stress, or buckling.

H3. RUPTURE OF FLANGES WITH HOLES SUBJECT TO TENSION

At locations of bolt holes in flanges subject to tension under combined axial force and major axis flexure, flange tensile rupture strength shall be limited by Equation H3-1. Each flange subject to tension due to axial force and flexure shall be checked separately.

$$\frac{P_r}{P_c}+\frac{M_{rx}}{M_{cx}} \leq 1.0 \qquad \text{(H3-1)}$$

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, kips (N)

P_c = available axial strength for the limit state of tensile rupture of the net section at the location of bolt holes, $\phi_t P_n$ or P_n/Ω_t, 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, kip-in. (N-mm)

M_{cx} = available flexural strength about x-axis for the limit state of tensile rupture of the flange, $\phi_b M_n$ or M_n/Ω_b, determined in accordance with Section F11.1. 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

The design of composite members is permitted by rational analysis subject to approval by the authority having jurisdiction.

User Note: Discussion of some aspects of the design of composite structural stainless steel and structural concrete members is given in the Commentary.

CHAPTER J

DESIGN OF CONNECTIONS

This chapter addresses connecting elements, connectors, and the affected elements of connected austenitic and duplex stainless steel members not subject to fatigue loads. It also addresses precipitation hardening connectors.

The chapter is organized as follows:

J1. General Provisions
J2. Welds
J3. Bolts and Threaded Parts
J4. Affected Elements of Members and Connecting Elements
J5. Fillers
J6. Splices
J7. Bearing Strength
J8. Pins
J9. Column Bases and Bearing on Concrete
J10. Anchor Rods and Embedments
J11. Doubly Symmetric I-Shaped Members with Concentrated Forces
J12. Square and Rectangular HSS with Concentrated Forces

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 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 Section A3.1c, by complete-joint-penetration (CJP) groove welds, the following provisions apply: (a) material notch-toughness requirements as given in Sections A3.1c; (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 M2.4. 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. Beam Copes and Weld Access Holes

All beam copes and weld access holes shall be free of notches or sharp reentrant corners. Beam cope radii and access holes shall provide a smooth transition past the points of tangency of adjacent surfaces and 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 size and shape of weld access holes shall be adequate for deposition of sound weld metal and provide clearance for weld tabs.

(c) The weld access hole shall have a length from the toe of the weld preparation not less than 1.5 times the thickness of the material in which the hole is made, nor less than 1½ in. (38 mm).

(d) Reentrant corners or cut materials shall be formed to provide a gradual transition with a minimum radius of 1 in. (25 mm). A radius of less than 1 in. (25 mm) is permitted if the 1-in. (25-mm) radius is unattainable due to the dimensions of the parts. The reentrant corner is permitted to be formed by mechanical cutting. Thermal cutting by plasma or laser is also permitted if at least ⅛ in. (3 mm) of material is mechanically removed from any cut edge. Both shall meet the surface requirements of Section M2.4.

(e) 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 weld access hole and weld access holes shall be free of notches and sharp reentrant corners.

(f) The weld access hole shall have a minimum radius of ⅜ in. (10 mm). A radius of less than ⅜-in. (10 mm) is permitted if the ⅜-in. (10-mm) radius is unattainable due to the dimensions of the parts.

(g) The weld 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 weld access hole.

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. Welded Alterations to Structures with Existing Bolts

In making welded alterations to structures, existing stainless steel bolts shall not be considered as sharing the load in combination with welds. The alteration shall be accomplished using either an all welded or all bolted connection.

J2. WELDS

All provisions in AWS D1.6/D1.6M apply to austenitic and duplex stainless steels under this Specification. Weld procedures shall be qualified in accordance with AWS B2.1/B2.1M. An AWS standard welding procedure specification (SWPS) based on AWS B2.1/B2.1M (AWS B2.1-X-XXX series) is also permitted.

User Note: Details of welds for stainless steel are generally similar to details of welds for other steel alloys. Further information is given in AISC Design Guide 27, *Structural Stainless Steel*.

User Note: Although they are regularly welded, AWS D1.6/D1.6M does not have a prequalified welding procedure specification (PWPS) for higher alloyed austenitic stainless steels (N08904, S31254, N08367, N08926) or the duplex stainless steels.

User Note: When the contract documents require corrosion testing of duplex stainless steel welds, Standard Test Method ASTM A1084 is used for the lean duplex steels S32101, S32202, and S32304. Standard Test Method ASTM A923 is used for the higher alloyed duplex steels S32003, S32205, S32760, S32750, and S82441.

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. No effective throat increase for weld reinforcement shall be allowed.

For prequalified austenitic stainless steel partial-joint-penetration (PJP) groove welds filled flush, the effective throat shall be determined as given in Table J2.1. For all flare groove welds filled flush, the weld shall be as shown 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.

For PJP groove welds with reinforcing fillet welds, the effective throat shall be the shortest distance from the joint root to the weld face of the diagrammatic weld minus ⅛ in. (3 mm) for any groove detail requiring such deduction as provided in Table J2.1. For flare-bevel-groove welds with reinforcing fillet welds, the effective throat

<table>
<tr><th colspan="4">TABLE J2.1
Prequalified Partial-Joint-Penetration
Groove Weld Effective Throat for
Austenitic Stainless Steels</th></tr>
<tr><th>Welding Process</th><th>Welding Position
F (flat),
H (horizontal),
V (vertical),
OH (overhead)</th><th>Groove Type
(AWS D1.6/AWS D1.6M,
Figure 5.3)</th><th>Effective Throat</th></tr>
<tr><td>Shielded metal arc (SMAW)</td><td rowspan="2">All</td><td rowspan="2">J or U groove
60° V</td><td rowspan="3">Depth of groove</td></tr>
<tr><td>Gas metal arc (GMAW)
Flux cored arc (FCAW)</td></tr>
<tr><td>Submerged arc (SAW)</td><td>F</td><td>J or U groove
60° bevel or V</td></tr>
<tr><td>Gas metal arc (GMAW)
Flux cored arc (FCAW)</td><td>F, H</td><td>45° bevel</td><td>Depth of groove</td></tr>
<tr><td>Shielded metal arc (SMAW)</td><td>All</td><td rowspan="2">45° bevel</td><td rowspan="2">Depth of groove
minus ⅛ in.
(3 mm)</td></tr>
<tr><td>Gas metal arc (GMAW)
Flux cored arc (FCAW)</td><td>V, OH</td></tr>
</table>

<table>
<tr><th colspan="2">TABLE J2.2
Effective Throat of Flare-Groove Welds</th></tr>
<tr><th>Flare-Bevel-Groove Welds</th><th>Flare-V-Groove Welds</th></tr>
<tr><td>(5⁄16)R</td><td>(½)R[a]</td></tr>
<tr><td colspan="2">[a] Use (3⁄8)R for GMAW. Effective size shall be qualified for the GMAW short circuiting transfer process.
Note: R = radius of outside surface
Source: AWS D1.6/D1.6M, Table 4.2 (see also AWS D1.6/D1.6M, clauses 4.4.1.2 and 4.4.2.2)</td></tr>
</table>

shall be the shortest distance from the joint root to the weld face of the diagrammatic weld minus the deduction for incomplete joint penetration.

Larger effective throats than those in Table J2.2 are permitted for a given welding procedure specification (WPS), provided the fabricator establishes by qualification the consistent production of such larger effective throat. Qualification shall consist of sectioning the weld normal to its axis, at mid-length, and terminal ends. Such sectioning shall be made on a number of combinations of material sizes representative of the range to be used in the fabrication. No weld size increase for penetration into the joint root or for weld reinforcement shall be allowed.

The maximum effective length of any groove weld, regardless of orientation, shall be the width of the part perpendicular to the direction of the tensile or compressive stress. For groove welds transmitting shear, the effective length is the length specified.

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. When a complete-joint-penetration groove weld is required and the welding will be from one side of the joint, there should be a root gap when welding higher alloyed austenitic (N08904, S31254, N08367, N08926) and duplex stainless steels.

1b. Limitations

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

2. Fillet Welds

2a. Effective Area

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

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

2b. Limitations

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 provisions do not apply to fillet weld reinforcements of PJP or CJP groove welds.

(b) The maximum size of fillet welds of connected parts shall be:

  (1) Along edges of material less than ¼ in. (6 mm) thick; not greater than the thickness of the material.

  (2) Along edges of material ¼ in. (6 mm) or more in thickness; not greater than the thickness of the material minus 1⁄16 in. (2 mm), unless the weld is especially designated on the drawings to be built out to obtain full-throat thickness. In the as-welded condition, the distance between the edge of the base metal and the toe of the weld is permitted to be less than 1⁄16 in. (2 mm), provided the weld size is clearly verifiable.

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 ¼ (6) inclusive	⅛ (3)
Over ¼ (6) to ½ (13)	3/16 (5)
Over ½ (13) to ¾ (19)	¼ (6)
Over ¾ (19) to 1½ (38)	5/16 (8)
Over 1½ (38) to 2¼ (56)	⅜ (10)
Over 2¼ (56) to 6 (150)	½ (13)
Over 6 (150)	⅝ (16)

[a] See Table J2.1.

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 ¼ (6) inclusive	⅛ (3)
Over ¼ (6) to ½ (13)	3/16 (5)
Over ½ (13) to ¾ (19)	¼ (6)
Over ¾ (19)	5/16 (8)

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

(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. The minimum length of an intermittent fillet weld segment shall be 1½ in. (38 mm) unless otherwise shown on approval drawings.

(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:

$$\beta = 1.2 - 0.002(l/w) \leq 1.0 \tag{J2-1}$$

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 $180w$.

(e) Intermittent fillet welds are permitted to be used to transfer calculated stress across a joint or faying surface 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 1½ in. (38 mm).

(f) In lap joints, the minimum amount of overlap in stress carrying lap joints shall be five times the thickness of the thinner part joined, but not less than 1 in. (25 mm). Lap joints in parts carrying axial stress shall be fillet welded along the end of both lapped parts, except where deflection of the joint is sufficiently restrained to prevent it from opening under load. Unless lateral deflection of the parts is prevented, they are to be connected by at least two transverse lines of plug or slot welds, or by two or more longitudinal welds.

(g) Unless otherwise specified, fillet welds need not start nor terminate less than the weld size from the end of the joint.

(h) For structures not cyclically loaded, fillet welds stressed by forces not parallel to the faying surface shall not terminate at corners of parts or members, but shall be returned continuously, full size, around the corner for a length equal to twice the weld size where such return can be made in the same plane. Weld returns shall be indicated on design and detail drawings where required.

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. (19 mm) 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.

(i) Fillet welds in holes or slots are permitted to be used to transmit shear and to prevent the buckling or separation of lapped parts. Fillet welds in holes or slots are not to be considered plug or slot welds. Sizes of holes and slots in which fillet welds are to be deposited shall be large enough to ensure that the fillet welds do not overlap, and base metal is visible between the weld toes. Should the fillet welds in holes or slots overlap, the welds shall be considered as partially filled plug or slot welds. 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 that extend to the edge of the part.

3. Plug and Slot Welds

3a. Effective Area

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

3b. Limitations

Plug or slot welds made by SMAW, GMAW, GTAW, and FCAW are permitted to be used to transmit shear in lap joints or to prevent buckling or separation of lapped parts, 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 2 mm), nor greater than the minimum diameter plus 1⁄8 in. (3 mm) or 2¼ 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 minimum width of the slot in which a slot weld is to be deposited shall be the thickness of the part in which it is made plus 5⁄16 in. (8 mm) or 2½ times the thickness of the member, whichever is smaller. The maximum width of the slot shall be the minimum plus 1⁄8 in. (3 mm) or 2¼ times the thickness of the weld, whichever is greater. The ends of the slot shall be semicircular.

(d) The minimum spacing of lines of slot welds in a direction transverse to their length shall be four times the width of the slot.

(e) The minimum center-to-center spacing in a longitudinal direction on any line shall be two times the length of the slot.

(f) The depth of the filling 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). The engineer of record (EOR) may specify an alternative limit of depth of the filling. In no case is the depth of filling required to be greater than the thickness of the thinner part being joined.

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} = cross-sectional 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.

In welded joints of cold-worked austenitic stainless steel, due to the effect of annealing in the heat affected zone, F_{nBM} shall be taken as the tensile strength of the annealed material, as given in the relevant ASTM standard for the product.

(b) For fillet welds, the available strength is permitted to be determined accounting for a directional strength increase of $(1.0 + 0.50\sin^{1.5}\theta)$ if strain compatibility of the various weld elements is considered,

where

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

(1) For a linear weld group with a uniform leg size, loaded through the center of gravity

$$R_n = F_{nw} A_{we} \tag{J2-4}$$

where

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

F_{EXX} = filler metal classification strength, ksi (MPa)

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

TABLE J2.5
Available Strength of Welded Joints, ksi (MPa)[a], [b]

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)
COMPLETE-JOINT-PENETRATION GROOVE WELDS				
Tension—Normal to weld axis	Base	Governed by J4		
	Weld	$\phi = 0.75$ $\Omega = 2.00$	F_{EXX}[c]	See J2.1a
Compression—Normal to weld axis	Base	$\phi = 0.90$ $\Omega = 1.67$	F_y	See J4
	Weld	$\phi = 0.75$ $\Omega = 2.00$	F_{EXX}[c]	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.			
Shear	Base	Governed by J4		
	Weld	$\phi = 0.75$ $\Omega = 2.00$	$0.60F_{EXX}$[c]	See J2.1a
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	$\phi = 0.80$ $\Omega = 1.88$	$0.60F_{EXX}$[c]	See J2.1a
Compression—Column to base plate and column splices designed per Section J1.4(a)	Compressive stress is permitted to be neglected in design of welds joining the parts.			
Compression—Connections of members designed to bear other than columns as described in Section J1.4(b)	Base	$\phi = 0.90$ $\Omega = 1.67$	F_y	See J4
	Weld	$\phi = 0.80$ $\Omega = 1.88$	$0.60F_{EXX}$[c]	See J2.1a
Compression—Connections not finished-to-bear	Base	$\phi = 0.90$ $\Omega = 1.67$	F_y	See J4
	Weld	$\phi = 0.80$ $\Omega = 1.88$	$0.90F_{EXX}$[c]	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.			

TABLE J2.5 (continued)
Available Strength of Welded Joints, ksi (MPa)[a], [b]

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)
Shear	Base	Governed by J4		
	Weld	$\phi = 0.75$ $\Omega = 2.00$	$0.60F_{EXX}$[c]	See J2.1a
FILLET WELDS, INCLUDING FILLETS IN HOLES AND SLOTS AND SKEWED T-JOINTS				
Shear	Base	Governed by J4		
	Weld	$\phi = 0.75$ $\Omega = 2.00$	$0.60F_{EXX}$[c] [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		
	Weld	$\phi = 0.75$ $\Omega = 2.00$	$0.60F_{EXX}$[c]	J2.3a

[a] The strength of laser and laser hybrid welds is equivalent to the base metal.
[b] See AWS D1.6/D1.6M, Table 5.3, for filler metals for matching strength to PWPS base metals in AWS D1.6/D1.6M, Table 5.2. For alloys without AWS D1.6/D1.6M PWPS, the producers and welding consumable manufacturers can advise if matching strength filler metals are available.
[c] Nominal tensile strength of filler metals for austenitic and duplex stainless steels, F_{EXX}, shall be determined from the relevant specification of AWS A5, Committee on Filler Metals and Allied Materials.
[d] The provisions of Section J2.4(b) are also applicable.

(2) 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) $R_n = 0.85R_{nwl} + 1.5R_{nwt}$ (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. Welding Consumable and Electrode Requirements

When welding consumables are used, the specified filler metal shall have welded corrosion resistance equivalent to or exceeding that of the base metal.

User Note: In some cases, the combination of base metal, prequalified filler metal, welding procedure specification, and desired corrosion resistance can create conditions where the filler metal has a lower strength than the base metal and will govern design of the connection.

When qualifying a weld procedure, if the filler metal has a lower strength than the base metal, mechanical testing shall be carried out in accordance with AWS D1.6/D1.6M, clause 6.9.3.3(2), and the tensile strength of the joint shall be greater than or equal to the specified minimum tensile strength of the filler metal. The filler metal shall be specified to ensure adequate corrosion resistance and strength.

User Note: AISC Design Guide 27, *Structural Stainless Steel*, gives a table of appropriate filler metals for common alloys of stainless steel and guidance on their corrosion resistance.

A complete list of austenitic stainless steels with base metal prequalification and PWPS filler metals is in AWS D1.6/D1.6M. Welding for highly alloyed austenitic stainless steels and duplex stainless steels is not prequalified so welding procedure specification (WPS) qualification is required. Guidance for filler metals should be obtained from the steel producer.

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.

8. Welding Dissimilar Steels

When welding dissimilar stainless steels, both corrosion resistance and strength matching shall be considered during design.

User Note: AWS D1.6/1.6M, Annex D, gives suggested filler metals for welding austenitic stainless steels to themselves or to other steels. When welding stainless steels to other steel alloys, the most important consideration is avoiding the formation of hard, brittle martensite in the weld joint and hot cracking. E309L, ER309L, E309LMo, or ER309LMo are commonly used to avoid these problems.

For welded joints between other steel alloys and stainless steels, any paint system applied to the other steel alloys shall extend over the bimetallic weldment onto the stainless steel to a distance of at least 2 in. (50 mm).

Any coating system applied to the other steel alloys shall be completely removed prior to any welding operation. In addition, coatings systems applied to the dissimilar weld shall be designed and applied to avoid any zinc or zinc components from contacting the stainless steel material and the weld.

User Note: Zinc can cause a phenomenon called solid metal embrittlement of stainless steel, resulting in potentially catastrophic failure.

J3. BOLTS AND THREADED PARTS

1. Stainless Steel Bolts

Use of stainless steel bolts shall conform to the following provisions of the RCSC *Specification for Structural Joints Using High-Strength Bolts*, hereafter referred to as the RCSC *Specification*, as approved by the Research Council on Structural Connections, with modifications listed:

- SECTION 1. GENERAL REQUIREMENTS: Sections 1.2, 1.3, 1.4, and 1.6.
- SECTION 2. FASTENER COMPONENTS: Sections 2.1, 2.2, 2.3.2 (except that structural bolt dimensions shall meet the requirements of this Specification), Table C-2.1, Table C-2.2, and Section 2.4.2 (except that nut dimensions shall meet the requirements of this Specification).
- SECTION 3. BOLTED PARTS: Sections 3.2.1 (uncoated faying surfaces only), 3.2.2 (1) (uncoated faying surfaces only), 3.3, and 3.4.
- SECTION 4. JOINT TYPE
- SECTION 7. PRE-INSTALLATION VERIFICATION: Except the minimum bolt pretension for pre-installation verification is given as $1.05T_b$, where T_b is given in Equation J3-5 and not by Table 7.1 of the RCSC *Specification*.
- SECTION 8. INSTALLATION: Sections 8.1, 8.2.1, and 8.2.2, except the minimum bolt pretensions in Table 8.1 are given in Equation J3-5 and the nut or head rotations in Table 8.2 do not apply to stainless steel bolts. Twist-off-type tension-control bolt pretensioning and direct tension-indicator pretensioning are outside the scope of this Specification.

- SECTION 9. INSPECTION: Sections 9.1, 9.2.1, 9.2.2, and 9.3, except that a pretension greater than that given by Equation J3-5 shall not be cause for rejection.
- SECTION 10. ARBITRATION: Except the pretension is given by Equation J3-5 and not Table 8.1.

Stainless steel bolts and nuts shall be in accordance with one of the ASTM standards listed in Section A3.3.

The dimensions of a bolting assembly shall comply with the following specifications:

Bolts: ASME B18.2.1
Bolts used in bearing-type connections shall have either hex heads or heavy hex heads. Bolts used in pretensioned or slip-critical connections shall have heavy hex heads. Threads shall be cut or rolled in accordance with ASME B1.1 unified coarse (UNC) or 8 thread series (8 UN), Class 2A.

Nuts: ASME B18.2.2
Nuts used in bearing-type connections shall be either hex or heavy hex. Nuts used in pretensioned or slip-critical connections shall be heavy hex. Threads shall be in accordance with ASME B1.1 UNC or 8 UN, Class 2B.

Washers: ASME B18.21.1

The bolts, washers, and nuts shall all have equivalent or greater corrosion resistance than the most corrosion resistant of the metal alloys joined.

User Note: The installation parameters used in the pre-installation verification may be developed using the bolt tightening qualification procedure provided in AISC Design Guide 27, *Structural Stainless Steel*.

Stainless steel bolts used in slip-critical and pretensioned applications shall not be reused after they have been tightened to their design preload. Stainless steel washer-type indicating devices and twist-off-type tension control bolt assemblies are outside the scope of this Specification. Other alternative-design fasteners, as described in RCSC *Specification* Section 2.8, are permitted if they meet the manufacturing, chemical composition, and mechanical property requirements for the alloys permitted in Section A3.1b and ASTM standards listed in Section A3.3 of this Specification.

The faying surface of slip-critical joints shall be grit blasted.

When assembled, all joint surfaces, including those adjacent to the washers, shall be free of scale, including tight mill scale, and cleaned in accordance with ASTM A380/A380M or A967/A967M, whichever is appropriate.

Galling shall be considered in design if the disassembly of bolted connections is a performance requirement.

User Note: When surfaces are under load and in relative motion, fastener thread galling may occur. Galling is more likely to occur in stainless steel bolting assemblies than in other steel alloy bolting assemblies. Galling can be avoided by taking the following measures:

- Lubricate the internal or external threads with products containing molybdenum disulfide, anti-seize products containing silver or copper powders, mica, graphite, or talc, or a suitable proprietary pressure wax.
- Reduce bolt tightening speed.
- Use of galling-resistant, high-silicon (e.g., S21800) stainless steels or alloys of different hardness for the bolt and nut.
- Make sure that the threads, as well as the bearing surfaces, are undamaged with no burrs.
- Keep the bolted interface clean and free of grit and abrasive materials.

(a) Bolts 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, 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* Section 4

  (2) Connections subject to vibratory loads where bolt loosening is a consideration

  (3) End connections of built-up members composed of two shapes interconnected by bolts, as required in Section E6.1

(c) Connections shall be designed as slip critical where required by the RCSC *Specification* Section 4.

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 Equation J3-5 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.

2. Size and Use of Holes

The following requirements apply for bolted connections:

(a) The maximum sizes of holes for bolts are given in Table J3.1 or Table J3.1M, except that larger holes, required for tolerance in the location of anchor rods in concrete foundations, are permitted in column base details.

(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 EOR.

(c) Finger shims up to ¼ 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
Nominal Hole Dimensions, in.

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

TABLE J3.1M
Nominal Hole Dimensions, mm

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

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

(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 made of stainless steel of equivalent corrosion resistance to the fasteners and nuts. They are not required for snug-tightened joints, except when the outer face of the joint has a slope greater than 1:20 with respect to a plane that is normal to the bolt axis or when a slotted hole occurs in an outer ply. Hardened stainless steel washers shall be used under both the bolt head and nut in pretensioned connections subject to fatigue loading, and slip-critical joints.

3. Minimum Spacing

The distance between centers of standard, oversized, or slotted holes shall not be less than 2⅔ 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.

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 J3.2 or Table J3.2M, 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 J3.3 or Table J3.3M.

User Note: The edge distances in Tables J3.2 and J3.2M 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 to the nearest edge of parts in contact shall be 12 times the thickness of the connected part under consideration, but shall not exceed 6 in. (150 mm). The longitudinal spacing of fasteners between elements consisting of a plate and a shape, or two plates, in continuous contact shall not exceed 24 times the thickness of the thinner part or 12 in. (300 mm).

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 bolt or pretensioned austenitic and duplex stainless steel bolt or threaded part shall be determined according to the limit states of tension rupture and shear rupture as:

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

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

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}, ksi (MPa)

$F_{nt} = 0.75F_u$

$F_{nv} = 0.45F_u$ if threads are not excluded from the shear planes

$= 0.55F_u$ if threads are excluded from the shear planes

TABLE J3.2
Minimum Edge Distance[a] from Center of Standard Hole[b] to Edge of Connected Part, in.

Bolt Diameter	Minimum Edge Distance
½	¾
⅝	⅞
¾	1
⅞	1⅛
1	1¼
1⅛	1½
1¼	1⅝
Over 1¼	1¼d

[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 EOR.
[b] For oversized or slotted holes, see Table J3.3.

TABLE J3.2M
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 EOR.
[b] For oversized or slotted holes, see Table J3.3M.

TABLE J3.3
Values of Edge Distance Increment, C_2, in.

<table>
<tr><th rowspan="3">Nominal Diameter of Fastener</th><th rowspan="3">Oversized Holes</th><th colspan="3">Slotted Holes</th></tr>
<tr><th colspan="2">Long Axis Perpendicular to Edge</th><th rowspan="2">Long Axis Parallel to Edge</th></tr>
<tr><th>Short Slots</th><th>Long Slots[a]</th></tr>
<tr><td>≤ ⅞</td><td>1/16</td><td>⅛</td><td rowspan="3">¾d</td><td rowspan="3">0</td></tr>
<tr><td>1</td><td>⅛</td><td>⅛</td></tr>
<tr><td>≥ 1⅛</td><td>⅛</td><td>3/16</td></tr>
</table>

[a] When the length of the slot is less than the maximum allowable (see Table J3.1), C_2 is permitted to be reduced by one-half the difference between the maximum and actual slot lengths.

TABLE J3.3M
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	0.75*d*	0
24	3	3		
≥ 27	3	5		

[a] When the length of the slot is less than the maximum allowable (see Table J3.1M), C_2 is permitted to be reduced by one-half the difference between the maximum and actual slot lengths.

The value for F_u shall be taken as the specified minimum tensile strength of the bolt given in the relevant ASTM standard.

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

Matching bolt/nut assemblies shall be used to preclude the possibility of failure by thread stripping, such as bolts in accordance with ASTM F593 used with nuts in accordance with ASTM F594.

The above design requirements shall also apply to precipitation hardening fasteners in accordance with ASTM F593 and ASTM A453/A453M but with the following resistance and safety factors:

$$\phi = 0.67 \text{ (LRFD)} \qquad \Omega = 2.25 \text{ (ASD)}$$

User Note: The force that can be resisted by a snug-tightened or pretensioned bolt or threaded part may be limited by the bearing or tearout strength of the material 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 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 subject to combined tension and shear shall be determined according to the limit states of tension and shear rupture as:

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

For austenitic and duplex stainless steel bolts:

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

For precipitation hardening stainless steel bolts:

$$\phi = 0.67 \text{ (LRFD)} \qquad \Omega = 2.25 \text{ (ASD)}$$

where

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

$$= 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_{rv} \leq F_{nt} \text{ (LRFD)} \quad \text{(J3-3a)}$$

$$= 1.3F_{nt} - \frac{\Omega F_{nt}}{F_{nv}} f_{rv} \leq F_{nt} \text{ (ASD)} \quad \text{(J3-3b)}$$

F_{nt} = nominal tensile stress from Section J3.6, ksi (MPa)

F_{nv} = nominal shear stress from Section J3.6, 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_r, 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_{rt}.

8. Stainless Steel Bolts in Slip-Critical Connections

Slip-critical connections are permitted using austenitic and duplex stainless steel bolts. 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 the design slip resistance.

For bolts used in slip-critical connections,

$$80 \text{ ksi (550 MPa)} \leq F_{yb} \leq 116 \text{ ksi (800 MPa)}$$

where

F_{yb} = specified minimum yield strength of bolt, ksi (MPa)

> **User Note:** The installation parameters used in the pre-installation verification may be developed using the bolt tightening qualification procedure provided in AISC Design Guide 27, *Structural Stainless Steel*. The bolt tightening qualification procedure given in AISC Design Guide 27, *Structural Stainless Steel*, includes suitability tests on the bolting assemblies to be used in the project. The test results are used to evaluate the strength, ductility, and lubrication of the bolting assemblies, and to determine the tightening parameters for the turn-of-nut, calibrated wrench, or combined installation methods.

The single bolt available slip resistance for the limit state of slip shall be determined as follows:

$$R_n = \mu D_u h_f T_b n_s \quad \text{(J3-4)}$$

(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 = 1.0, a multiplier that reflects the ratio of the mean installed bolt pretension to the specified minimum bolt pretension.

T_b = minimum fastener tension for stainless steel bolts, determined as:

$$= 0.7F_{yb}A_s \quad \text{(J3-5)}$$

A_s = net tensile area of bolt, in.^{2} (mm^2)

> **User Note:** The Commentary provides common values for the net tensile area of bolts.

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, given in Table J3.4

> **User Note:** If other faying surface types are employed, such as with a blast media other than grit or a different surface roughness, tests can be conducted according to the testing method given in AISC Design Guide 27, 2nd Edition, *Structural Stainless Steel*, to determine the slip coefficient of the potential faying surface.

9. Combined Tension and Shear in Slip-Critical Connections

When a slip-critical connection is subject 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:

$$k_{sc} = 1 - \frac{T_u}{D_u T_b n_b} \geq 0 \quad \text{(LRFD)} \qquad \text{(J3-6a)}$$

$$k_{sc} = 1 - \frac{1.5T_a}{D_u T_b n_b} \geq 0 \quad \text{(ASD)} \qquad \text{(J3-6b)}$$

where

T_a = required tension force using ASD load combinations, kips (N)

T_u = required tension force using LRFD load combinations, kips (N)

n_b = number of bolts carrying the applied tension

TABLE J3.4
Slip Coefficients, μ, for Friction Surfaces

Class[a]	Slip coefficient, μ[b]
SSB	0.20
SSC	0.40
SSD	0.50

[a] Surface classes are defined in Section M2.13.
[b] The potential loss of preloading force due to time dependent relaxation from its initial value is considered in these slip coefficient values.

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:

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

The nominal strength of the connected material, R_n, is determined as follows:

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

(1) Bearing

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

$$R_n = 1.25dtF_u \tag{J3-7a}$$

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

For $l_2/d_h > 1.5$

$$R_n = 2.5dtF_u \tag{J3-7b}$$

For $l_2/d_h \le 1.5$

$$R_n = 2.0dtF_u \tag{J3-7c}$$

User Note: The use of Equation J3-7b may lead to the occurrence of plastic deformation under service loads.

(2) Tearout

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

$$R_n = 1.25\left(\frac{l_1}{2d_h}\right)dtF_u \tag{J3-7d}$$

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

$$R_n = 2.5\left(\frac{l_1}{3d_h}\right)dtF_u \qquad \text{(J3-7e)}$$

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

(1) Bearing

$$R_n = 1.04dtF_u \qquad \text{(J3-7f)}$$

(2) Tearout

$$R_n = 1.04\left(\frac{l_1}{2d_h}\right)dtF_u \qquad \text{(J3-7g)}$$

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

where

F_u = specified minimum tensile strength of the connected material, ksi (MPa)

d = nominal diameter of fastener, in. (mm)

d_h = diameter of hole, in. (mm)

l_1 = half of the distance between the center of the hole and the center of the adjacent hole or distance between the center of the hole and the edge of the material, in the direction of the force, in. (mm)

l_2 = half of the distance between the center of the hole and the center of the adjacent hole or distance between the center of the hole and the edge of the material, in the direction perpendicular to the force, in. (mm)

t = design thickness of connected material, as defined in Section B4.2, 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 included in the relevant ASTM standard shall be verified by tests. These suitability tests shall include the verification of the ductility of the bolting assembly as well as the friction in the paired threads and the bearing surfaces. Furthermore, the achievable bolt force level should be evaluated. If the fasteners are to be pretensioned, relaxation tests shall also be carried out.

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.

13. Bolting Dissimilar Metals

The use of other steel alloy bolts, including galvanized bolts, with stainless steel elements shall not be allowed.

In a bolted joint involving stainless steel in combination with one or more dissimilar metals that may become wet from rain, fog, spray, occasional or regular immersion, high humidity, or condensation, the metals shall be electrically isolated to prevent galvanic and crevice corrosion. The method of isolation shall be appropriate for the type of exposure and shall not permit moisture infiltration into the joint, particularly in immersed or otherwise regularly wet applications. It shall also accommodate any differences in the thermal movement of the connected materials.

When insulating washers and bushings are used to provide corrosion protection in snug-tightened connections, the product used in the bolt grip shall be specified by the EOR.

> **User Note:** The amount of tightening required in the connection should be provided in the design documents, based on recommendations of the isolation material manufacturer. Further information about bolting dissimilar metals is given in AISC Design Guide 27, 2nd Edition, *Structural Stainless Steel*.

J4. AFFECTED ELEMENTS OF MEMBERS AND CONNECTING ELEMENTS

This section applies to elements of austenitic and duplex stainless steel members at connections and connecting elements, such as plates, gussets, angles, and brackets; all of them made of austenitic or duplex stainless steel.

1. Strength of Elements in Tension

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

(a) For tensile yielding of connecting elements

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

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

(b) For tensile rupture of connecting elements

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

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

where

A_e = effective net area as defined in Section D3, in.^{2} (mm^2)

User Note: The effective net area of the connection plate may be limited due to stress distribution as calculated by methods such as the Whitmore section.

User Note: To restrict irreversible deformations in bolted connections, the stresses at the net cross section of the connecting material at bolt holes should be limited to a stress smaller than F_u, as described in Section D2.

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

$$R_n = 0.60 C_v F_y A_{gv} \tag{J4-3}$$

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

where

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

C_v = shear buckling strength coefficient
= 1.2

(b) For shear rupture of the element

$$R_n = 0.60 F_u A_{nv} \tag{J4-4}$$

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

where

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

3. Block Shear Strength

The available strength for the limit state of block shear rupture along a shear failure path or paths and a perpendicular tension failure path shall be determined as follows:

$$R_n = 0.60 F_u A_{nv} + U_{bs} F_u A_{nt} \le 0.60 C_v F_y A_{gv} + U_{bs} F_u A_{nt} \tag{J4-5}$$

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

where

A_{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, $U_{bs} = 0.5$.

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 in accordance with the provisions of Chapter E,

where

L_c = effective length of connecting element, in. (mm)
= KL
K = effective length factor
L = laterally unbraced length of connecting element, 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. General

Fillers shall be made of a stainless steel with equivalent corrosion resistance and strength to that of the structure.

2. 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 J5.2a or Section J5.2b, as applicable.

2a. Thin Fillers

Fillers less than ¼ in. (6 mm) thick shall not be used to transfer stress. When the thickness of the fillers is less than ¼ in. (6 mm), or when the thickness of the filler is ¼ 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.

2b. 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 area subject 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.

3. Fillers in Bolted Bearing-Type Connections

When a bolt that carries load passes through fillers that are equal to or less than ¼ 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 ¼ in. (6 mm) thick, one of the following requirements shall apply:

(a) The shear strength of the bolts shall be multiplied by the factor

$$1 - 0.4(t - 0.25)$$

$$1 - 0.0154(t - 6) \text{ (S.I.)}$$

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 beams shall develop the nominal strength of the smaller spliced section. Other types of splices in cross sections of 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:

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

For finished surfaces; pins in reamed, drilled, or bored holes; and ends of fitted bearing stiffeners, the nominal bearing strength, R_n, shall be determined as follows:

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

where

A_{pb} = projected area in bearing, in.^{2} (mm^2)

F_y = specified minimum yield stress, ksi (MPa)

J8. PINS

This section applies to pins that are used in pin-connected members, as addressed in Section D5.

1. Bearing Strength

The design bearing strength, ϕR_n, and the allowable bearing strength, R_n/Ω, of pins shall be determined in accordance with Section J7.

2. Shear Strength

The design shear strength, ϕR_n, and the allowable shear strength, R_n/Ω, of pins shall be determined as follows:

$$R_n = 0.6F_u A_p \tag{J8-1}$$

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

where

A_p = gross area of the pin, in.^{2} (mm^2)
F_u = specified minimum tensile strength of the pin, ksi (MPa)

3. Flexural Strength

The design flexural strength, ϕM_n, and the allowable flexural strength, M_n/Ω, of pins shall be determined as follows:

$$M_n = 1.5F_y S \tag{J8-2}$$

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

where

F_y = specified minimum yield stress of the pin, ksi (MPa)
S = elastic section modulus of the pin, in.^{3} (mm^3)

The moment in a pin shall be calculated on the basis that it is simply supported by the connected parts.

User Note: For a typical pin-connected assembly, the Commentary gives a method for calculating the maximum moment in a pin by representing the load transfer between the pin and the mating parts as concentrated forces.

4. Combined Shear and Flexure

The interaction of shear and flexure in pins shall be limited by Equation J8-3:

$$\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{M_r}{M_c}\right)^2 \leq 1.0 \tag{J8-3}$$

where

V_r = required shear strength, determined in accordance with Chapter C, using LRFD or ASD load combinations, kips (N)
V_c = available shear strength, ϕR_n or R_n/Ω, determined in accordance with Section J8.2, kips (N)
M_r = required flexural strength, ϕM_n or M_n/Ω, determined in accordance with Chapter C, using LRFD or ASD load combinations, kip-in. (N-mm)
M_c = available flexural strength, determined in accordance with Section J8.3, kip-in. (N-mm)

J9. 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:

$$\phi_c = 0.65 \text{ (LRFD)} \qquad \Omega_c = 2.31 \text{ (ASD)}$$

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

(a) On the full area of a concrete support

$$P_p = 0.85 f_c' A_1 \tag{J9-1}$$

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

$$P_p = 0.85 f_c' A_1 \sqrt{A_2 / A_1} \leq 1.7 f_c' A_1 \tag{J9-2}$$

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)

J10. 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 of Section J3.6.

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*, 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 washers or plate washers to bridge the hole.

> **User Note:** There is no separate ASTM standard covering stainless steel anchor rods; they should be specified in accordance with one of the ASTM standards listed in Section A3.4.

User Note: See ACI 318 (ACI 318M) for embedment design and for shear friction design. OSHA requirements for eccentric loads on columns during steel erection may affect anchor rod design.

J11. DOUBLY SYMMETRIC I-SHAPED MEMBERS WITH CONCENTRATED FORCES

This section applies to single- and double-concentrated forces applied normal to the flange(s) of doubly symmetric I-shaped members. 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 J11.8. Doublers shall also meet the design requirements in Section J11.9.

User Note: See Appendix 6, Section 6.3, for requirements for the ends of cantilever members.

Stiffeners shall be provided at unframed ends of beams in accordance with the requirements of Section J11.7.

1. Flange Local Bending

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

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

$$R_n = 6.25 F_{yf} t_f^2 \tag{J11-1}$$

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

where

F_{yf} = specified minimum yield stress of the flange, ksi (MPa)

t_f = design thickness of the loaded flange, as defined in Section B4.2, 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 J11-1 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:

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

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

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

$$R_n = F_{yw} t_w \left(5k + l_b\right) \tag{J11-2}$$

(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

$$R_n = F_{yw} t_w \left(2.5k + l_b\right) \tag{J11-3}$$

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 for rolled sections, or the thickness of the flange for welded sections, in. (mm)

l_b = length of bearing (not less than k for end beam reactions), in. (mm)

t_w = design thickness of web, as defined in Section B4.2, 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 $d/2$

$$R_n = 0.80 t_w^2 \left[1 + 3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{E F_{yw} t_f}{t_w}} \tag{J11-4}$$

(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 $l_b/d \le 0.2$

$$R_n = 0.40 t_w^2 \left[1 + 3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}} \tag{J11-5a}$$

(2) For $l_b/d > 0.2$

$$R_n = 0.40 t_w^2 \left[1 + \left(\frac{4l_b}{d} - 0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}} \tag{J11-5b}$$

where

E = modulus of elasticity, ksi (MPa)

d = full nominal depth of the member, in. (mm)

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:

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

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

(a) If the compression flange is restrained against rotation

(1) When $(h/t_w)/(L_b/b_f) \le 2.3$

$$R_n = \frac{C_r t_w^3 t_f}{h^2}\left[1 + 0.4\left(\frac{h/t_w}{L_b/b_f}\right)^3\right] \tag{J11-6}$$

(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 $(h/t_w)/(L_b/b_f) \le 1.7$

$$R_n = \frac{C_r t_w^3 t_f}{h^2}\left[0.4\left(\frac{h/t_w}{L_b/b_f}\right)^3\right] \tag{J11-7}$$

(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 J11-6 and J11-7, the following definitions apply:

C_r = 960,000 ksi (6.6×10^6 MPa), when $M_u < M_y$ (LRFD) or $1.5M_a < M_y$ (ASD) at the location of the force
= 480,000 ksi (3.3×10^6 MPa), when $M_u < M_y$ (LRFD) or $1.5M_a < M_y$ (ASD) at the location of the force

L_b = largest laterally unbraced length along either flange at the point of load, in. (mm)

M_a = required flexural strength using ASD load combinations, kip-in. (N-mm)

M_u = required flexural strength using LRFD 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)

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:

$$R_n = \frac{24t_w^3\sqrt{EF_{yw}}}{h} \tag{J11-8}$$

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

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

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

6. Web Panel-Zone Shear

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

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

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

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

(a) When the effect of inelastic panel-zone deformation on frame stability is not accounted for in the analysis:

(1) For $\alpha P_r \leq 0.4P_y$

$$R_n = 0.60F_y d_c t_w \tag{J11-9}$$

(2) For $\alpha P_r > 0.4P_y$

$$R_n = 0.60F_y d_c t_w \left(1.4 - \frac{\alpha P_r}{P_y}\right) \tag{J11-10}$$

(b) When the effect of inelastic panel-zone deformation on frame stability is accounted for in the analysis:

(1) For $\alpha P_r \leq 0.75P_y$

$$R_n = 0.60F_y d_c t_w \left(1 + \frac{3b_{cf} t_{cf}^2}{d_b d_c t_w}\right) \tag{J11-11}$$

(2) For $\alpha P_r > 0.75P_y$

$$R_n = 0.60F_y d_c t_w \left(1 + \frac{3b_{cf} t_{cf}^2}{d_b d_c t_w}\right)\left(1.9 - \frac{1.2\alpha P_r}{P_y}\right) \tag{J11-12}$$

In Equations J11-9 through J11-12, 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} = design thickness of column flange, as defined in Section B4.2, in. (mm)
t_w = design thickness of column web, as defined in Section B4.2, 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 J11.9 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 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 $0.75h$, a cross section composed of two stiffeners, and a strip of the web having a width of $0.94t_w\sqrt{E/F_y}$ at interior stiffeners and $0.47t_w\sqrt{E/F_y}$ 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 J11.3, J11.5, and J11.7.

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 J11.6) 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.

J12. SQUARE AND RECTANGULAR HSS WITH CONCENTRATED FORCES

This section applies to compressive single- and double-concentrated forces applied normal to the flange(s) of square and rectangular HSS.

The design strength, ϕR_n, and the allowable strength, R_n/Ω, for the limit state of web local yielding, web local crippling, and web compression buckling of square and rectangular HSS under compressive concentrated forces shall be determined as follows:

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

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

$$R_n = Ct^2F_y\left(1 - C_r\sqrt{\frac{r}{t}}\right)\left(1 + C_l\sqrt{\frac{l_b}{t}}\right)\left(1 - C_h\sqrt{\frac{h}{t}}\right) \tag{J12-1}$$

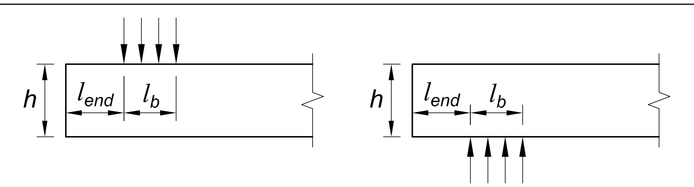
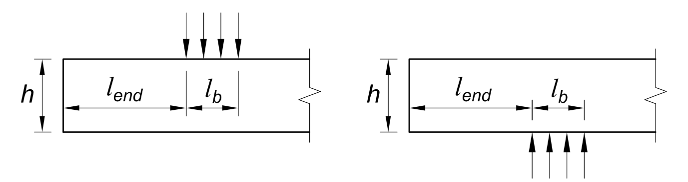
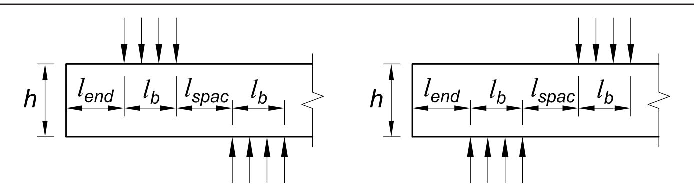
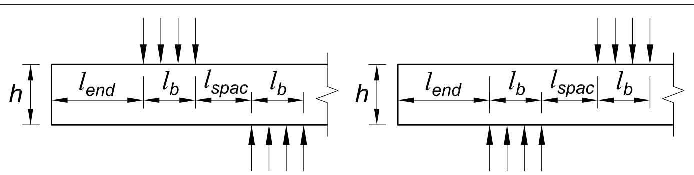
where

C = coefficient from Table J12.1
C_h = web slenderness coefficient from Table J12.1
C_l = bearing length coefficient from Table J12.1
C_r = internal bend radius coefficient from Table J12.1
F_y = specified minimum yield stress, ksi (MPa)
h = clear distance between flanges less the corner radius, in. (mm)
l_b = length of bearing, in. (mm)
r = internal radius of corner, which may be taken as $2t$ if not known, in. (mm)
t = design wall thickness, as defined in Section B4.2, in. (mm)

TABLE J12.1
Coefficients for Square and Rectangular HSS

Case	C	C_r	C_l	C_h
End one-flange loading (EOF)	4	0.32	1.60	0.040
Interior one-flange loading (IOF)	2	0.04	2.30	0.001
End two-flange loading (ETF)	2	0.35	2.60	0.050
Interior two-flange loading (ITF)	8	0.21	0.75	0.010

TABLE J12.2
Loading Cases for Square and Rectangular HSS

Case	Illustrations	
EOF		$l_{end} \le 1.5h$
IOF		$l_{end} > 1.5h$
ETF		$l_{end} \le 1.5h$ $l_{spac} \le 1.5h$
ITF		$l_{end} > 1.5h$ $l_{spac} \le 1.5h$

End one-flange loading (EOF): distance from the bearing edge to the end of the member ≤ 1.5h, and the clear distance between the bearing edges of adjacent opposite concentrated forces or reactions > 1.5h.

Interior one-flange loading (IOF): distance from the bearing edge to the end of the member > 1.5h, and the clear distance between the bearing edges of adjacent opposite concentrated forces or reactions > 1.5h.

End two-flange loading (ETF): distance from the bearing edge to the end of the member ≤ 1.5h, and the clear distance between the bearing edges of adjacent opposite concentrated forces or reactions ≤ 1.5h.

Interior two-flange loading (ITF): distance from the bearing edge to the end of the member > 1.5h, and the clear distance between the bearing edges of adjacent opposite concentrated forces or reactions ≤ 1.5h.

Table J12.1 shall apply to square and rectangular HSS subject to the loading cases defined in Table J12.2, when the following limitations are satisfied:

(a) $h/t \le 60$

(b) $l_b/t \le 55$

(c) $l_b/h \le 3$

(d) $r/t \le 2$

CHAPTER K

ADDITIONAL REQUIREMENTS FOR HSS AND BOX-SECTION CONNECTIONS

This chapter addresses additional requirements for connections to austenitic and duplex stainless steel square or round HSS members and square box sections of uniform thickness, where seam welds between box-section elements are complete-joint-penetration (CJP) groove welds in the connection region. The provisions are only applicable to truss connections. The requirements of Chapter J also apply.

The chapter is organized as follows:

User Note: The design of moment connections to square, rectangular, and round HSS members, and truss connections to rectangular HSS members is outside the scope of this chapter due to insufficient research and test data to substantiate the design of these types of connections.

K1. GENERAL PROVISIONS

For the purposes of this chapter, the centerlines of branch members and chord members shall lie in a common plane. Square 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 only those 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.

User Note: Connection strength is often governed by the size of HSS members, especially the 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.

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

K2. HSS-TO-HSS TRUSS CONNECTIONS

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

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

A_g = gross area of member, in.^{2} (mm^2)
B = overall width of square HSS main member, measured 90° to the plane of the connection, in. (mm)
B_b = overall width of square HSS branch member, measured 90° to the plane of the connection, in. (mm)
B_e = effective width of square HSS branch member, 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, ksi (MPa)
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_{end} = distance from the near side of the connecting branch to end of chord, 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)
t = design wall thickness of HSS main member, as defined in Section B4.2, in. (mm)
t_b = design wall thickness of HSS branch member, as defined in Section B4.2, 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 square 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
γ = chord slenderness ratio; the ratio of one-half the diameter to the thickness, $D/2t$, for round HSS; the ratio of one-half the width to thickness, $B/2t$, for square HSS
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 square HSS

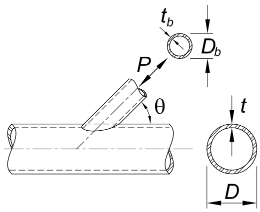
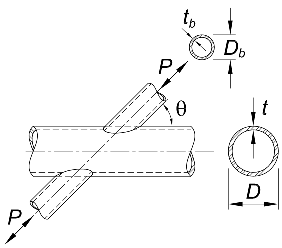
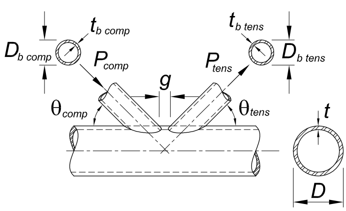
2. Round HSS

The available strength of round HSS-to-HSS truss connections, within the limits in Table K2.1a, shall be taken as the lowest value obtained according to the limit states shown in Table K2.1.

User Note: Some round HSS-to-HSS cross-connections may exceed the maximum deformation criteria of 1% of the chord diameter, D, at serviceability limit states associated with the strength equations given in Table K2.1. Therefore, if deformations at service loads are critical for these types of joints, the design should be based on a more detailed analysis.

User Note: For connections outside the range of validity given in Table K2.1a, a detailed analysis should be made. This analysis should take account of the secondary moments in the joints caused by the bending stiffness of the joints.

TABLE K2.1
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 $D_{b\,(tens/comp)} < (D - 2t)$	Limit state: shear yielding (punching) $P_n = 0.6 F_y t \pi D_b \left(\dfrac{1 + \sin\theta}{2\sin^2\theta} \right)$ (K2-2) $\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)
T- and Y-connections 	Limit state: chord plastification $P_n \sin\theta = F_y t^2 \left(3.1 + 15.6\beta^2\right)\gamma^{0.2} Q_f$ (K2-3) $\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)
Cross-connections 	Limit state: chord plastification $P_n \sin\theta = F_y t^2 \left(\dfrac{5.7}{1 - 0.81\beta} \right) Q_f$ (K2-4) $\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)
K-connections with gap or overlap 	Limit state: chord plastification $(P_n \sin\theta)_{compression\ branch} = F_y t^2 \left(2.0 + 11.33 \dfrac{D_{b\,comp}}{D} \right) Q_g Q_f$ (K2-5) $(P_n \sin\theta)_{tension\ branch} = (P_n \sin\theta)_{compression\ branch}$ (K2-6) $\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)

3. Square HSS

The available strength, ϕP_n and P_n/Ω, of square HSS-to-HSS truss connections within the limits in Table K2.2a, shall be taken as the lowest value obtained according to limit states shown in Table K2.2 and Chapter J.

> **User Note:** Outside the limits in Table K2.2a, the limit states of Chapter J are still applicable and the applicable limit states of Chapter K are not defined.

TABLE K2.1 (continued)
Available Strengths of Round HSS-to-HSS Truss Connections

Functions

Q_f = 1 for chord (connecting surface) in tension
= 1.0 − 0.3U (1 + U) for HSS (connecting surface) in compression (K2-7)

$$U = \left| \frac{P_{ro}}{F_c A_g} + \frac{M_{ro}}{F_c S} \right| \tag{K2-8}$$

where P_{ro} and M_{ro} are determined on the side of the joint that has the lower compression stress. P_{ro} and M_{ro} refer to required strengths in the HSS: $P_{ro} = P_u$ for LRFD and $P_{ro} = P_a$ for ASD; $M_{ro} = M_u$ for LRFD and $M_{ro} = M_a$ for ASD.

$$Q_g = \gamma^{0.2} \left[1 + \frac{0.024\gamma^{1.2}}{\exp\left(\dfrac{0.5g}{t} - 1.33 \right) + 1} \right] \tag{K2-9}$$

Note that exp(x) is equal to e^x, where e is the base of the natural logarithm.

TABLE K2.1a
Limits of Applicability of Table K2.1

Joint eccentricity:	−0.55	$\le e/D \le 0.25$ for K-connections
Chord wall slenderness:	D/t	≤ 50 for T-, Y-, and K-connections
	D/t	≤ 40 for cross-connections
Branch wall slenderness:	D_b/t_b	≤ 50 for tension and compression branch
	D_b/t_b	$\le 0.05E/F_{yb}$ for compression branch
Width ratio:	0.2	$< D_b/D \le 1.0$ for T-, Y-, cross-, and overlapped K-connections
	0.4	$\le D_b/D \le 1.0$ for gapped K-connections
Gap:	g	$\ge t_{b\ comp} + t_{b\ tens}$ 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
End distance:	l_{end}	$\ge D\left(1.25 - \frac{\beta}{2}\right)$ for T-, Y-, cross-, and K-connections

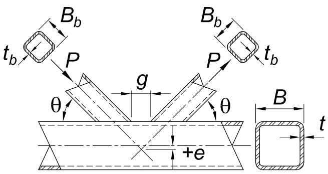
3a. Effective Width for Connections to Square HSS

The effective width of square HSS branches perpendicular to the longitudinal axis of a square HSS member that deliver a force component transverse to the face of the member shall be taken as:

$$B_e = \left(\frac{10t}{B} \right) \left(\frac{F_y t}{F_{yb} t_b} \right) B_b \le B_b \tag{K2-1}$$

<table>
<tr><th colspan="2">TABLE K2.2
Available Strengths of Square
HSS-to-HSS Truss Connections</th></tr>
<tr><th>Connection Type</th><th>Connection Available Axial Strength</th></tr>
<tr><td>

Gapped K-connections



</td><td>

Limit state: chord wall plastification, for all β

$$P_n \sin\theta = F_y t^2 \left(9.8\beta_{eff}\gamma^{0.5}\right) Q_f \qquad \text{(K2-10)}$$

$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)

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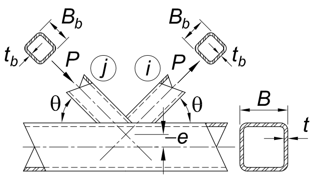
<tr><td>

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

$\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)

When $25\% \le O_v < 50\%$

$$P_{n,i} = F_{ybi} t_{bi} \left[\frac{O_v}{50} \left(2B_{bi} - 4t_{bi}\right) + B_{ei} + B_{ej} \right] \qquad \text{(K2-11)}$$

When $50\% \le O_v < 80\%$

$$P_{n,i} = F_{ybi} t_{bi} \left(2B_{bi} - 4t_{bi} + B_{ei} + B_{ej}\right) \qquad \text{(K2-12)}$$

When $80\% \le O_v \le 100\%$

$$P_{n,i} = F_{ybi} t_{bi} \left(2B_{bi} - 4t_{bi} + B_{bi} + B_{ej}\right) \qquad \text{(K2-13)}$$

Subscript *i* refers to the overlapping branch
Subscript *j* refers to the overlapped branch

$$P_{n,j} = P_{n,i} \left(\frac{F_{ybj} A_{bj}}{F_{ybi} A_{bi}} \right) \qquad \text{(K2-14)}$$

</td></tr>
<tr><th colspan="2">Functions</th></tr>
<tr><td colspan="2">

$Q_f = 1$ for chord (connecting surface) in tension

$$= 1.3 - 0.4 \frac{U}{\beta_{eff}} \le 1.0 \qquad \text{(K2-15)}$$

$$U = \left| \frac{P_{ro}}{F_c A_g} + \frac{M_{ro}}{F_c S} \right| \qquad \text{(K2-8)}$$

where P_{ro} and M_{ro} are determined on the side of the joint that has the lower compression stress.
P_{ro} and M_{ro} refer to required strengths in the HSS:
$P_{ro} = P_u$ for LRFD and $P_{ro} = P_a$ for ASD; $M_{ro} = M_u$ for LRFD and $M_{ro} = M_a$ for ASD.

$$\beta_{eff} = \left[(B_b)_{compression\ branch} + (B_b)_{tension\ branch} \right] / 2B \qquad \text{(K2-16)}$$

</td></tr>
</table>

TABLE K2.2a
Limits of Applicability of Table K2.2

Joint eccentricity:	-0.55	$\leq e/B \leq 0.25$ for K-connections
Chord wall slenderness:	B/t	≤ 35 for gapped K-connections
	B/t	≤ 30 for overlapped K-connections
Branch wall slenderness:	B_b/t_b	≤ 35 for tension branch
		$\leq 1.25\sqrt{\dfrac{E}{F_{yb}}}$ for compression branch of gapped K-connections
		≤ 35 for compression branch of gapped K-connections
		$\leq 1.1\sqrt{\dfrac{E}{F_{yb}}}$ for compression branch of overlapped K-connections
Width ratio:	B_b/B	≥ 0.25 for overlapped K-connections
Overlap:	25%	$\leq O_v \leq$ 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
Additional Limits for Gapped K-Connections		
Width ratio:	$\dfrac{B_b}{B} \geq 0.1 + \dfrac{\gamma}{50}$	
	$\dfrac{B_b}{B} \geq 0.35$	
Gap ratio:	$\zeta = g/B \geq 0.5\left(1-\beta_{eff}\right)$	
	$\zeta = g/B \leq 1.5\left(1-\beta_{eff}\right)$	
Gap:	$g \geq t_{b\ compression\ branch} + t_{b\ tension\ branch}$	
Branch size:	smaller $B_b \geq 0.63$ (larger B_b)	

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

User Note: Stainless steel is usually specified because of its corrosion resistance or for aesthetic reasons. Section A4 gives minimum alloy requirements for specific service environments. The Commentary lists the information that is typically necessary for appropriate alloy specification. AISC Design Guide 27, 2nd Edition, *Structural Stainless Steel*, gives further guidance on the assessment and importance of those service environment characteristics in selection of an appropriate alloy and surface finish in order to achieve the design life of the structure.

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 ASCE *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.

Deflections shall be determined for the load combination at the relevant serviceability limit state. Unless a more exact method is used, standard structural theory is permitted for estimating the deflection of elastic beams, except that the modulus of elasticity shall be replaced with the reduced modulus of elasticity, E_r, determined in accordance with Equation L2-1:

$$E_r = \frac{E_{st} + E_{sc}}{2} \tag{L2-1}$$

where

E_{sc} = secant modulus corresponding to the maximum compressive stress in the cross section, which may be determined in accordance with Appendix 7, Equation A-7-5, ksi (MPa)

E_{st} = secant modulus corresponding to the maximum tensile stress in the cross section, which may be determined in accordance with Appendix 7, Equation A-7-5, ksi (MPa)

User Note: Replacing the modulus of elasticity by the reduced modulus provides accurate predictions of the deflection when the maximum stress in the cross section does not exceed 65% of F_y. At higher levels of stress, the method becomes very conservative.

Table User Note L2.1 gives the secant modulus for common types of stainless steel at a maximum stress in the cross section equal to $0.6F_y$, which may be conservatively adopted in preliminary estimates of deflection.

TABLE USER NOTE L2.1
Secant Modulus

Stainless Steel		F_y, ksi (MPa)	E_S, ksi (MPa)
Austenitic	S30400 and S31600	30 (205)	25,800 (177 000)
	S30403 and S31603	25 (170)	25,400 (175 000)
Duplex	S32101, S32202, S32205, and S82011	65 (450)	28,300 (195 000)
	S32304	58 (400)	28,200 (194 000)

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.

CHAPTER M
FABRICATION AND ERECTION

This chapter addresses requirements for fabrication and erection documents, fabrication, and erection.

The chapter is organized as follows:

M1. Fabrication and Erection Documents
M2. Fabrication
M3. Erection

M1. FABRICATION AND ERECTION DOCUMENTS

Fabrication and erection documents are permitted to be prepared in stages. Fabrication documents shall be prepared in advance of fabrication and give complete information necessary for the fabrication and alloy traceability of the component parts of the structure, including the location, type, and size of welds and bolts. The necessary treatment for bimetallic interface connections between the stainless steel and other steel alloy components must be specified. Erection documents shall be prepared in advance of installation and give information necessary for erection of the structure. Fabrication and erection documents shall clearly distinguish between shop and field welds and bolts. They shall clearly identify pretensioned and slip-critical bolted connections. Fabrication and erection documents shall be made with due regard to speed and economy in fabrication and erection.

M2. FABRICATION

1. Identification

Hard stamped, punched, or drilled marks are not permitted for stainless steel. Soft or low stress stamps, or stamped metal tags attached by stainless steel wire, may be used. Temporary color coding or marking with paint, crayon, ink stencil, or similar approved chloride and sulfide free marking products is permitted during fabrication, but shall be fully removed prior to on-site welded fabrication and project completion.

2. Handling and Storage

Contractors shall be required during all stages of handling, shipment, storage, and erection to take preventative measures to avoid surface finish damage, free iron, or other material contamination due to exposure to steel cutting or grinding, and other potential sources of damage that may adversely affect appearance or performance.

Contact with chemicals and acids, including dyes, glues, adhesive tape, hydrochloric acid, chloride containing cleaning products, and undue amounts of oil and grease shall be avoided. If it is necessary to use them, their suitability and the maximum duration of exposure shall be determined.

User Note: See AISC Design Guide 27, 2nd Edition, *Structural Stainless Steel*, and ASTM A967/A967M for information on chemical passivation methods for removing embedded metals and other surface contamination from stainless steels to restore corrosion resistance. Deeply embedded metal contamination may need to be removed by chemical descaling or another acceptable method in accordance with ASTM A380/A380M.

3. Cambering, Curving, and Straightening

Mechanical means, such as stretcher- or tension-leveling, may be specified to introduce or correct camber, curvature, and straightness. Stretcher- or tension-leveling tolerances shall be specified in accordance with the applicable general requirements in the ASTM standards listed in Section A3 for the product or as agreed in the contract documents.

If the temperature and exposure time cannot be carefully controlled, warm forming or straightening shall not be permitted. Unless otherwise indicated by the alloy producer or when there will be solution annealing after forming, the maximum austenitic stainless steel temperature for warm forming or straightening shall be 900°F (480°C). For duplex stainless steel the maximum allowable temperature for warm forming or straightening is 750°F (400°C). If the working temperature exceeds 750°F (400°C), a full solution anneal and rapid quench of the duplex stainless steel shall be required. Annealing colors and oxide scales shall be completely removed in accordance with ASTM A380/A380M.

User Note: Duplex stainless steels have very low strength at annealing temperatures, and distortion is likely during annealing.

4. Cutting

Oxyacetylene cutting shall not be used for cutting stainless steel without a powder fluxing technique and is strongly discouraged. If it is used, then the resulting surface shall be cleaned in accordance with AWS D1.6/1.6M, clause 7.20, Weld Cleaning.

Shearing, sawing, abrasive cutting, water jet cutting, and thermal cutting by plasma or laser are permitted for structural stainless steel. Thermal cutting shall not be used unless at least ⅛ in. (3 mm) of material is mechanically removed from any thermally cut edge. Oxyacetylene torch cutting shall not be used.

Notches or gouges on cut surfaces (edges) not exceeding 1⁄16 in. (2 mm) for materials less than ⅝ in. (16 mm) in thickness or 10% of the material thickness for materials ⅝ in. (16 mm) or greater need not be repaired unless specified by the engineer of record (EOR) or contract specifications. If fatigue service is anticipated, the EOR shall establish limits for notches and gouges in areas with stress concentrations.

Notches, gouges, or other material discontinuities may be repaired by grinding or machining provided the depth of the notch or gouge does not exceed the lesser of ⅛ in. (3 mm) or 20% of the material thickness, and shall be blended smoothly into the surrounding surfaces to a slope not exceeding 1 in. in 4 in. (25 mm in 100 mm). Notches or gouges exceeding this size shall be repaired by excavation and welding in accordance with AWS D1.6/D1.6M, clause 7.5, unless otherwise directed by the EOR. Repaired surfaces shall be cleaned to bright metal after completing the repair.

Reentrant corners shall be formed with a curved transition. The radius need not exceed that required to fit the connection. The requirements of AWS D1.6/D1.6M, clause 8, Part C, Acceptance Criteria, shall be used to evaluate discontinuities and determine if repair is required.

Beam copes and weld access holes shall meet the requirements given in Section J1.6.

5. Planing of Edges

Planing or finishing of sheared or 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.

6. Welded Construction

Welding shall be performed in accordance with AWS D1.6/D1.6M, except as modified in Section J2. Welding consumables that produce weld deposits of at least equivalent corrosion resistance to the parent metal shall be used.

If the weld is not fully shielded to prevent heat tint and scale formation, the corrosion resistance of the weld shall be restored by chemical descaling or another suitable method in accordance with ASTM A380/A380M. The contract documents shall specify a cleaning procedure in accordance with ASTM A380/A380M that is suitable for the corrosiveness of the service environment. Chemical passivation and electropolishing are not permissible methods of restoring weld corrosion resistance unless combined with methods suitable for removing the chromium depleted layer.

User Note: The need for back-side shielding of welds should be considered.

General cleanliness and the absence of contamination shall be required for good weld quality. Oils, dirt, plastic film, and wax crayon marks shall be removed in accordance with ASTM A967/A967M or A380/A380M to avoid weld contamination.

User Note: Duplex stainless steel requires careful control and monitoring of the heat input and interpass temperature during welding and, in rare cases, requires postweld annealing per the material specification. An appropriate filler metal in accordance with AWS D1.6/D1.6M or the alloy producer's recommendation if no guidance is given by AWS D1.6/D1.6M should be used. Verification of the postweld heat treatment through corrosion testing or other means should be considered unless the product or process has been prequalified.

> **User Note**: Weld distortion is generally greater in stainless steel than in other steel alloys, particularly with austenitic stainless steel which has a higher coefficient of thermal expansion. Heat input and interpass temperatures should be controlled to minimize distortion and avoid potential metallurgical problems, and follow an appropriate weld procedure approved by the EOR.

Bimetallic interfaces in welded joints combining stainless steels with other steel alloys shall be protected from galvanic corrosion by a waterproof coating suitable for the service environment and intended maintenance frequency, unless the environment has consistently low humidity levels and no exposure to moisture from rain, fog, immersion, high humidity, and condensation. The coating shall consist of an epoxy coating, metal primer, and paint system, or other durable waterproof coating that is applied to the other steel alloy and extends over the weldment onto the stainless steel to a distance of at least 2 in. (50 mm).

If stainless steel will be painted, the passive film shall be abrasively or chemically removed immediately before application of the metal primer or a suitable etchant primer shall be used.

> **User Note:** The passive film on stainless steel fully reforms after 24 hours and its presence will cause poor paint adherence.

7. 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.

Machined, drilled, or water jet cut holes are permitted. Thermal cutting of holes by plasma or laser shall not be used unless at least ⅛ in. (3 mm) of material is mechanically removed from any cut edge. Gouges shall not exceed a depth of 1⁄16 in. (2 mm). If fatigue service is anticipated, the EOR shall establish limits for notches and gouges in areas with stress concentrations.

Fully inserted finger shims, with a total thickness of not more than ¼ 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 bolts shall conform to the requirements of Section J3.

> **User Note:** The lubrication of bolts is beneficial but corrosion arising from contamination of pre-applied lubrication by chloride salts, industrial pollutants, or other corrosive substances and metallic particles during storage may occur. This contamination could cause corrosion that might not otherwise occur, and may lead to premature failure. Application of lubricants shortly before use is advisable.

8. 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.

9. Dimensional Tolerances

Dimensional tolerances for fabrication shall be in accordance with AISC *Code of Standard Practice for Structural Stainless Steel Buildings* Chapter 6. Dimensional tolerances for structural shapes shall be specified in accordance with Table A3.1.

10. 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.

11. Holes for Anchor Rods

Holes for anchor rods shall be cut in accordance with Section M2.7.

12. 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.

13. Faying Surfaces for Slip-Critical Bolted Connections

Faying surfaces of slip-critical bolted connections shall be grit-blasted and shall have a defined surface roughness, as specified in Table M2.1, using either Rz or Rt. Every production faying surface does not require surface roughness inspection; however, a blast process shall be qualified to produce the required surface roughness and production faying surfaces shall require intermittent inspection. SSPC PA 17 shall be

TABLE M2.1
Definition of Surface Classes for Slip-Critical Faying Surfaces

Class	*Rz*[a]		*Rt*[b]	
	μin.	μm	μin.	μm
SSB	≥ 1400	≥ 35	≥ 2000	≥ 50
SSC	≥ 1800	≥ 45	≥ 2400	≥ 60
SSD	≥ 2200	≥ 55	≥ 2800	≥ 70

[a] *Rz* is the surface roughness according to ASTM D7127.
[b] *Rt* is the surface roughness according to ASTM D4417.

used for development of a qualified blast process and production inspection of the faying surfaces. Appendices A and B of SSPC PA 17 shall be mandatory with the following additional requirements in SSPC PA 17, Appendix B, Section B1:

> The representative sample shall have a minimum thickness of at least ¼ in. (6 mm) and shall be made of the same stainless steel material as the faying surfaces in the project. A copy of the information contained in SSPC PA 17 Tables B1 or B3 shall be submitted to the engineer of record for approval.

Clean stainless steel grit media shall be used when blast cleaning these surfaces to avoid contamination.

> **User Note:** The use of brand new media is advisable due to the likelihood of contamination by free iron embedded into the surface being blasted when recycled media is used.

M3. 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 AISC *Code of Standard Practice for Structural Stainless Steel Buildings* Chapter 7.

2. Stability and Connections

The frame of structural steel buildings shall be carried up true and plumb within the limits defined in AISC *Code of Standard Practice for Structural Stainless Steel Buildings* 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 AISC *Code of Standard Practice for Structural Stainless Steel Buildings*, 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 ¼ 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 shall be made of a stainless steel with similar or better durability than that of the structure.

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. Cleaning after Erection

Contamination of stainless steel by contact with free iron shall be avoided. Other steel alloy brushes, wool, scrapers, or other products shall not be used. To limit surface contamination, the stainless steel shall either be protected by removable plastic film or another barrier, and final cleaning shall be performed after completion of the structure in accordance with ASTM A967/A967M. Cleaning procedures shall be appropriate for the material, surface finish, function of the component, and corrosion risk, and shall not contain chlorides or hydrochloric acid. The method, level, and extent of cleaning shall be specified.

User Note: Some stainless steel products like bar, HSS, and pipe contain higher levels of sulfur. Chemical descaling by the manufacturer removes surface sulfides. If the surface finish is subsequently disturbed by finishing, machining, welding, or some other process that removes metal, additional sulfides will be exposed and the stainless steel will be more susceptible to corrosion. In corrosive environments, chemical passivation in accordance with ASTM A967/A967M is advisable after the last processing step that disturbs the surface to remove surface sulfides and achieve maximum corrosion resistance.

CHAPTER N

QUALITY CONTROL AND QUALITY ASSURANCE

This chapter addresses minimum requirements for quality control, quality assurance, and nondestructive testing for structural stainless steel systems 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) Surface preparations or coatings

The chapter is organized as follows:

N1. General Provisions
N2. Fabricator and Erector Quality Control Program
N3. Fabricator and Erector Documents
N4. Inspection and Nondestructive Testing Personnel
N5. Minimum Requirements for Inspection of Structural Stainless Steel Buildings
N6. Approved Fabricators and Erectors
N7. Nonconforming Material and Workmanship

N1. GENERAL PROVISIONS

Quality control (QC) as specified in this chapter shall be provided by the fabricator and erector. Quality assurance (QA) as specified in this chapter shall be provided by others when required by the authority having jurisdiction (AHJ), applicable building code, purchaser, owner, or engineer of record (EOR). 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 stainless 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 stainless steel building construction. There may be cases where supplemental inspections are advisable, including weld inspection in accordance with AWS D1.6/D1.6M. 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 stainless 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 this Specification, referenced ASTM standards, if applicable, and AISC *Code of Standard Practice for Structural Stainless Steel Buildings* Section 6.1, 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, bolting, and details in accordance with Section N5
(b) Handling of material in accordance with Section M2.2
(c) Shop cut and finished surfaces in accordance with Section M2.4
(d) Shop heating for cambering, curving, and straightening in accordance with Section M2.3
(e) Tolerances for shop fabrication in accordance with AISC *Code of Standard Practice for Structural Stainless Steel Buildings* Section 6.4. Dimensional tolerances for structural shapes shall be specified in accordance with Table A3.1.

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, bolting, and details in accordance with Section N5
(b) Handling of material in accordance with Section M2.2
(c) Steel deck in accordance with SDI *Standard for Quality Control and Quality Assurance for Installation of Steel Deck*
(d) Field cut surfaces in accordance with Section M2.4
(e) Field heating for straightening in accordance with Section M2.3
(f) Tolerances for field erection in accordance with AISC *Code of Standard Practice for Structural Stainless Steel Buildings* Section 7.13

N3. FABRICATOR AND ERECTOR DOCUMENTS

1. Submittals for Stainless Steel Construction

The fabricator or erector shall submit the following documents for review by the EOR or the EOR's designee, in accordance with AISC *Code of Standard Practice for Structural Stainless Steel Buildings*, 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 Stainless 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 stainless steel elements, copies of material test reports and certifications in accordance with Section A3.1.

(b) For stainless steel castings and forgings, copies of material test reports and certifications in accordance with Section A3.2.

(c) For fasteners, copies of manufacturer's test reports and 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 A3.6.

(f) 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.

(g) Welding procedure specifications (WPS) or documentation of prequalified welding procedure specifications (PWPS).

(h) Procedure qualification records (PQR) for WPS that are not prequalified in accordance with AWS D1.6/D1.6M.

(i) Welding personnel performance qualification records (WPQR) for the stainless steel alloy and WPS and continuity records.

(j) 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

(k) Fabricator's or erector's, as applicable, QCI qualifications.

(l) 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) with the addition in AWS B5.1, clause 7, that testing for fundamental knowledge of stainless be required, or

(b) Qualified under the provisions of AWS D1.6/D1.6M, clause 8.

QC bolting inspection personnel shall be qualified on the basis of documented training and experience in structural bolting inspection for both other steel alloy and stainless steel fasteners, including the issues related to bimetallic contact between dissimilar metals.

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) with the addition in clause 7 of AWS B5.1 that testing for fundamental knowledge of stainless be required, 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 D1.6/D1.6M, clause 8.1.4.

QA bolting inspection personnel shall be qualified on the basis of documented training and experience in structural bolting inspection for fasteners made from stainless steel and other steel alloys including the issues related to bimetallic interface contact between dissimilar metals.

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 D1.6/D1.6M, clause 8.1.4, 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), or

(c) Qualification and certification of NDT personnel (ISO 9712).

(d) Alternatively, performance-based qualification programs, in accordance with ASME ANDE-1 *ASME Nondestructive Examination and Quality Control Central Qualification and Certification Program*, may be used for training, examination, and certification activities as specified in the employer's written practice.

N5. MINIMUM REQUIREMENTS FOR INSPECTION OF STRUCTURAL STAINLESS 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 AISC *Code of Standard Practice for Structural Stainless Steel Buildings* 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 M2.6.

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 and shall be documented including the part inspected, date inspected, and results of the inspection.

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 D1.6/D1.6M.

> **User Note:** The technique, workmanship, appearance, and quality of welded construction is addressed in Section M2.6.

> **User Note:** MT is not an acceptable inspection method for austenitic stainless steels due to their nonmagnetic properties.

> **User Note:** Final inspection of austenitic and duplex stainless steels may begin immediately after welds have cooled to ambient temperature.

> **User Note:** Ultrasonic methods are of limited use on welds because of difficulties in interpretation; however, they can be used on parent material.

The following are required for UT:

(a) A weld mockup with reference reflectors placed in the weld is required to establish the validity of the inspection technique.

(b) The use of angle beam longitudinal wave transducers.

(c) A procedure demonstration acceptable to the EOR.

> **User Note:** Special weld preparation may be required as angle beam longitudinal wave transducers can only be used as a first leg inspection. The sound beam will not "bounce up" into the weld.

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
Verify handling procedures to avoid contamination	O	O
Fit-up of groove welds (including joint geometry) • Joint preparations • Dimensions (alignment, joint root opening, joint 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 K-joints without backing (including joint geometry) • Joint preparations • Dimensions (alignment, joint root opening, joint 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 joint 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 EOR and shall be the low-stress type and compatible with the stainless steel base metal.

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

5b. CJP Groove Weld NDT

For structures in risk category III or IV as determined from ASCE/SEI 7, 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 Subject 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.

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 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	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 built-up heavy shapes (see Section A3.1c) are welded, visually inspect the weld access hole for cracks.

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% if the rejection rate (the number of welds containing unacceptable defects divided by the number of welds completed) exceeds 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, location in the piece, and date tested.

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 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 applicable provisions of the RCSC *Specification* in accordance with Section J3.

(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 method with matchmarking techniques, 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 calibrated wrench method or the turn-of-nut method without matchmarking, 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.

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 applicable provisions of the RCSC *Specification* in accordance with Section J3, progressing systematically from the most rigid point toward the free edges	O	O

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. These tasks shall be performed for each joint or member and shall be documented including the part inspected, date inspected, and results of the inspection.

TABLE N5.6-3
Inspection Tasks After Bolting

Inspection Tasks After Bolting	QC	QA
Document acceptance or rejection of bolted connections	P	P

7. Other Inspection Tasks

The fabricator's QCI shall inspect the fabricated structural stainless 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 structural stainless 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 (other steel alloys or stainless steel) and other embedments supporting other structural steel alloys or structural stainless 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 structural stainless steel or erected structural stainless 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

QA inspection is permitted to be waived when the work is performed in a fabricating shop or by an erector approved by the AHJ to perform the work without QA.

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 QA agency shall review the fabricator's NDT reports.

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

APPENDIX 1
DESIGN BY ADVANCED ANALYSIS

This appendix permits the use of more 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 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.

> **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, as defined in Section F2.2. 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 on the 2016 AISC *Specification* 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 Structural Stainless Steel Buildings* 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 AISC *Code of Standard Practice for Structural Stainless Steel Buildings*. For out-of-straightness of members (member imperfections), a $1/1000$ out-of-straightness ratio is often considered.

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. The stiffness reduction factors,

τ_g 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 in this appendix.

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 $\tau_g\tau_b$, 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. Alternatively, the cross-section strength is permitted to be determined using the continuous strength method (CSM), following the provisions of Appendix 2, but with $\Lambda = 5$ (i.e., the maximum strain is limited to 5 times the yield strain).

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 inelastic analysis with the CSM strain limits.

The inelastic (plastic zone, distributed plasticity) analysis using the nonlinear material stress-strain model given in Appendix 7, 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) CSM strain limits; 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.

Traditional plastic hinge analysis is not appropriate, and an inelastic analysis, incorporating a nonlinear material stress-strain response, must be carried out for stainless steel design, as outlined in the following sections.

The provisions of this method apply only to doubly symmetric members, including I-shapes, HSS, and box sections.

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 inelastic deformation.

2a. Material

The specified minimum yield stress, F_y, of members subject to inelastic deformation shall not exceed 80 ksi (550 MPa).

2b. Unbraced Length

In prismatic member segments that experience significant plasticity ($M_r > M_p$, where M_r is the required strength and M_p is F_yZ), 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:

$$L_{pd} = \left(1.83 - 1.17\frac{M_1'}{M_2}\right)L_p \qquad \text{(A-1-1)}$$

where

L_p = limiting laterally unbraced length for the limit state of yielding, as defined by Equation F2-7, in. (mm)

(1) When the magnitude of the bending moment at any location within the unbraced length exceeds M_2

$$M_1'/M_2 = +1 \tag{A-1-2a}$$

Otherwise:

(2) When $M_{mid} \le (M_1 + M_2)/2$

$$M_1' = M_1 \tag{A-1-2b}$$

(3) When $M_{mid} > (M_1 + M_2)/2$

$$M_1' = (2M_{mid} - M_2) < M_2 \tag{A-1-2c}$$

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 shapes bent about their major axis

$$L_{pd} = \left(0.57 - 0.36\frac{M_1'}{M_2}\right)\frac{Et^2}{F_y d} \tag{A-1-3}$$

where

E = modulus of elasticity of stainless steel, ksi (MPa)
= 28,000 ksi (193 000 MPa) for austenitic, and 29,000 ksi (200 000 MPa) for duplex stainless steel

F_y = specified minimum yield stress, ksi (MPa)

d = depth of section, in. (mm)

t = design wall thickness of HSS as defined in Section B4.2, in. (mm)

For all types of members subject to axial compression and containing inelastic deformation, 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, where r_x and r_y are the radii of gyration about the major and minor axes, respectively.

There is no L_{pd} limit for member segments 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

2c. Axial Force

To ensure ductility in compression members with inelastic deformation, the design strength in compression shall not exceed $0.75F_yA_g$, where A_g is the gross area of the member.

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.

The structural analysis shall be carried out using finite element analysis with beam elements. In the analysis, the beam finite elements shall have a maximum length equal to the elastic local buckling half-wavelength of the full cross section, L_{el}.

> **User Note:** The local buckling half-wavelength of the full cross section, L_{el}, may be determined numerically or, for I-shaped sections and square and rectangular HSS, using the expressions given in the Commentary.

Strain limits, defined by the continuous strength method and outlined in Section 1.3.3d, shall be applied to the compressive strains of all cross sections in the structural system to simulate local buckling and control the extent to which spread of plasticity, moment redistribution, and strain hardening are exploited. The compressive strains may be averaged over the elastic local buckling half-wavelength, L_{el}, to allow for the beneficial effect of the local moment gradient. If L_{el} is not exactly divisible by the length of the finite elements, the strains should be averaged over the number of whole finite elements within L_{el}.

A cross section can withstand the required strain demands if the design strain, ε_r, is less than or equal to the CSM strain limit, ε_{csm}, at that location, as discussed in Section 1.3.3d.

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 plastic strength of the member cross section shall be represented in the analysis by explicit modeling of the nonlinear material stress-strain response. The stress-strain curve with strain hardening may be modeled in accordance with Equation A-7-1.

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.

> **User Note:** As an alternative to explicitly modeling residual stresses, an enhanced geometric imperfection magnitude may be utilized—see the Commentary.

3d. Continuous Strength Method Strain Limits

Strain limits are used in conjunction with second-order inelastic analysis to verify the capacity of the structure. For all cross sections, the following requirement shall be met:

$$\frac{\varepsilon_r}{\varepsilon_y} \le \frac{\varepsilon_{csm}}{\varepsilon_y} \tag{A-1-4}$$

Where ε_r is the required compressive strain, averaged over the local buckling half-wavelength, L_{el}, $\varepsilon_y = F_y/E$ is the yield strain, and the ratio $\varepsilon_{csm}/\varepsilon_y$ shall be determined using Equations A-1-5 and A-1-6.

> **User Note:** The required strain is determined at the outer compressive fiber and may be averaged over the local buckling half-wavelength of the full cross section. Note that the presence of stiffeners locally constrains the shape of the cross section and hence the local buckling half-wavelength is located to either side of the stiffener—see Commentary for further explanation.

When $\lambda_l \le 0.68$

$$\frac{\varepsilon_{csm}}{\varepsilon_y} = \frac{0.25}{\lambda_l^{3.6}} + \frac{0.002}{\varepsilon_y} \le \Lambda \tag{A-1-5}$$

When $0.68 < \lambda_l < 1.00$

$$\frac{\varepsilon_{csm}}{\varepsilon_y} = \left(1 - \frac{0.222}{\lambda_l^{1.05}}\right)\frac{1}{\lambda_l^{1.05}} + \frac{0.002\left(f/F_y\right)^n}{\varepsilon_y} \tag{A-1-6}$$

where

F_y = specified minimum yield stress, ksi (MPa)
f = stress at outer compressive fiber, ksi (MPa)
Λ = upper bound strain limit with a recommended value of 15
ε_{csm} = CSM strain limit
ε_y = F_y/E

λ_l = local cross-section slenderness

$$= \sqrt{\frac{F_y}{F_{el}}} \tag{A-1-7}$$

F_{el} = elastic local buckling stress of full cross section, ksi (MPa)

User Note: The elastic local buckling stress of the full cross section may be obtained numerically or, for I-shaped sections and square and rectangular HSS, using the expressions given in the Commentary.

3e. Shear, Torsion, and Bending Interaction

The interaction between bending, torsion, and shear shall be accounted for through a reduction factor, ρ_{csm}, applied to the CSM strain limit, ε_{csm}, as given by Equation A-1-8:

$$\frac{\varepsilon_r}{\rho_{csm}\varepsilon_{csm}} \leq 1.0 \tag{A-1-8}$$

where ρ_{csm} shall be determined as follows:

When $\dfrac{V_r}{V_c} + \dfrac{T_r}{T_c} \leq 0.5$

$$\rho_{csm} = 1.0 \tag{A-1-9}$$

When $\dfrac{V_r}{V_c} + \dfrac{T_r}{T_c} > 0.5$

$$\rho_{csm} = \frac{0.5}{0.5 + \rho} \tag{A-1-10}$$

where

V_r = required shear strength, determined in accordance with Section 1.3, using LRFD load combinations, kips (N)

V_c = available shear strength, $\phi_v V_n$, determined in accordance with Chapter G, kips (N)

T_r = required torsional strength, determined in accordance with Section 1.3, using LRFD load combinations, kip-in. (N-mm)

T_c = available torsional strength, $\phi_T T_n$, determined in accordance with Chapter G, kip-in. (N-mm)

$$\rho = \left[2\left(\frac{V_r}{V_c} + \frac{T_r}{T_c}\right) - 1\right]^2 \tag{A-1-11}$$

In addition, Equations A-1-12 and A-1-13 shall be satisfied.

$$V_r \leq V_c \tag{A-1-12}$$

$$T_r \leq T_c \tag{A-1-13}$$

APPENDIX 2

THE CONTINUOUS STRENGTH METHOD

This appendix presents an alternative design method for determining the strength of austenitic and duplex stainless steel doubly symmetric I-shaped members, HSS members, and doubly symmetric box sections of uniform thickness in tension, compression, flexure, and combined flexure and compression, and channels, angles, and tees in tension, compression, and flexure, according to the limit states of yielding and local buckling.

The appendix is organized as follows:

2.1. Limitations
2.2. Material Modeling
2.3. Deformation Capacity
2.4. Tensile Strength
2.5. Compressive Strength
2.6. Flexural Strength
2.7. Combined Flexure and Compression

2.1. LIMITATIONS

The continuous strength method (CSM) only applies to I-shapes, channels, angles, tees, HSS, and box sections that satisfy the slenderness limitations given in Table A-2.1.1. There is no maximum slenderness limit for members subject to axial tension. The method only applies to static design at ambient temperatures.

2.2. MATERIAL MODELING

The elastic, linear hardening material model used with the CSM is shown in Figure A-2.2.1,

where

E_{sh} = strain hardening modulus, ksi (MPa)

$$= \frac{F_u - F_y}{0.16\left(1 - F_y/F_u\right) - \varepsilon_y} \quad \text{(A-2-1)}$$

F_u = specified minimum tensile strength, ksi (MPa)

ε_y = yield strain

$$= F_y/E \quad \text{(A-2-2)}$$

2.3. DEFORMATION CAPACITY

The normalized deformation capacity, $\varepsilon_{csm}/\varepsilon_y$, shall be determined as follows.

TABLE A-2.1.1
Range of Applicability of the CSM

Description of Element	Compression Members	Flexural Members	Combined Loading
I-shapes, channels, angles, tees, square and rectangular HSS, and box sections	$\frac{L_c}{r} \le 0.63\sqrt{\frac{E}{F_y}}$ [a] or $\frac{F_y}{F_e} \le 0.04$ [a] and $\lambda_l \le 1.6$	$L_b \le 0.50L_p$ [a] and $\lambda_l \le 1.6$	Requirements for both compression and flexural members shall be satisfied
Round HSS	$\frac{L_c}{r} \le 0.63\sqrt{\frac{E}{F_y}}$ [a] or $\frac{F_y}{F_e} \le 0.04$ [a] and $\lambda_l \le 0.6$	$\lambda_l \le 0.6$	Requirements for both compression and flexural members shall be satisfied

[a] Limitations on member length shall not apply when designing according to Appendix 1.

where
- E = modulus of elasticity of stainless steel, ksi (MPa)
  = 28,000 ksi (193 000 MPa) for austenitic, and 29,000 ksi (200 000 MPa) for duplex stainless steel
- F_e = elastic buckling stress determined in accordance with Chapter E, ksi (MPa)
- F_y = specified minimum yield stress, which for cold-formed HSS, may be replaced with $F_{y,avg}$ determined in accordance with Section B4.3, ksi (MPa)
- L_b = length between points that are either braced against lateral displacement of the compression flange or between points braced to prevent twist of the cross section, in. (mm)
- L_c = effective length of member, as defined in Section E2, in. (mm)
- L_p = limiting laterally unbraced length for the limit state of yielding, as defined in Section F2, in. (mm)
- r = radius of gyration, in. (mm)
- λ_l = cross-section slenderness, as defined in Section 2.3.2

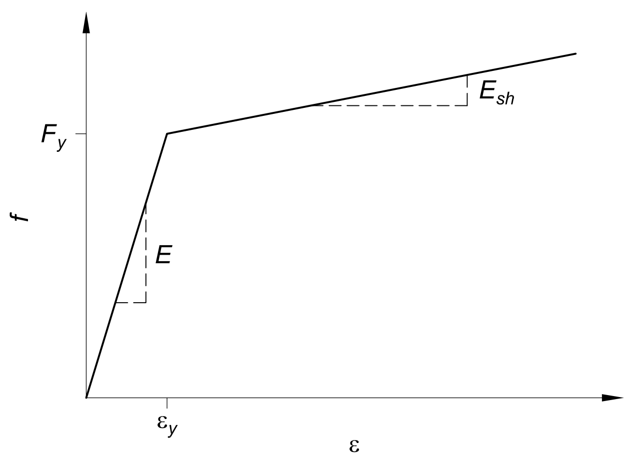


Fig. A-2.2.1. CSM material model.

1. I-Shapes, Channels, Angles, Tees, Square and Rectangular HSS, and Box Sections

(a) When $\lambda_l \le 0.68$

$$\frac{\varepsilon_{csm}}{\varepsilon_y} = \frac{0.25}{\lambda_l^{3.6}} \le \text{minimum}\left(\Lambda,\ \frac{0.10\left(1 - F_y/F_u\right)}{\varepsilon_y}\right) \qquad \text{(A-2-3)}$$

(b) When $\lambda_l > 0.68$

$$\frac{\varepsilon_{csm}}{\varepsilon_y} = \left(1 - \frac{0.222}{\lambda_l^{1.05}}\right)\frac{1}{\lambda_l^{1.05}} \qquad \text{(A-2-4)}$$

> **User Note:** The CSM strain limit is given by Equations A-2-3 and A-2-4 when the bilinear material model defined in Section 2.2 is used and by Equations A-1-5 and A-1-6 when the rounded two-stage Ramberg-Osgood material model is used in Appendix 1, Section 1.3.

2. Round HSS

(a) When $\lambda_l \le 0.30$

$$\frac{\varepsilon_{csm}}{\varepsilon_y} = \frac{4.44 \times 10^{-3}}{\lambda_l^{4.5}} \le \text{minimum}\left(\Lambda,\ \frac{0.10\left(1 - F_y/F_u\right)}{\varepsilon_y}\right) \qquad \text{(A-2-5)}$$

(b) When $\lambda_l > 0.30$

$$\frac{\varepsilon_{csm}}{\varepsilon_y} = \left(1 - \frac{0.224}{\lambda_l^{0.342}}\right)\frac{1}{\lambda_l^{0.342}} \qquad \text{(A-2-6)}$$

where

ε_{csm} = cross-section failure strain

λ_l = cross-section slenderness, determined according to Equation A-2-7

$$= \sqrt{\frac{F_y}{F_{el}}} \qquad \text{(A-2-7)}$$

F_{el} = elastic local buckling stress of full cross section, ksi (MPa)

Λ = upper bound strain limit; for use in Appendix 1, Section 1.2, $\Lambda = 5$. In all other cases, $\Lambda = 15$.

> **User Note:** The elastic local buckling stress may be obtained numerically or, for I-shaped members and square and rectangular HSS, using the expressions given in the Commentary.
>
> Alternatively, F_{el} may be determined using the simple equations given in this User Note. However, these equations may lead to very conservative estimations (especially the expression for sections comprised of flat plates, because it is based on the most slender constituent element of the cross section).

For I-shaped members, channels, angles, tees, rectangular HSS, and box sections

$$F_{el} = \frac{k\pi^2 E t^2}{12\left(1-\nu^2\right)b^2}$$

For round HSS

$$F_{el} = \frac{E}{\sqrt{3\left(1-\nu^2\right)}}\frac{2t}{D}$$

where

D = cross-section diameter, in. (mm)
b = width of the most slender element of the cross section, in. (mm)
t = design thickness of the most slender element for I-shaped members, channels, angles, tees, square and rectangular HSS, and box sections, and design wall thickness for round HSS, in. (mm)
ν = Poisson's ratio
= 0.3
k = plate buckling coefficient
= 0.425 for unstiffened compression elements
= 4.00 for stiffened compression elements
= 23.9 for stiffened elements subject to flexure

2.4. TENSILE STRENGTH

The design CSM tensile strength, $\phi_t P_n$, and the allowable CSM tensile strength, P_n/Ω_t, of austenitic and duplex stainless steel 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

$$P_n = f_{csm,t} A_g \tag{A-2-8}$$

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

(b) For tensile rupture in the net section

$$P_n = F_u A_e \tag{A-2-9}$$

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

where

A_e = effective net area, in.^{2} (mm^2)
A_g = gross area of member, in.^{2} (mm^2)
F_u = specified minimum tensile strength, ksi (MPa)
$f_{csm,t}$ = stress corresponding to $\varepsilon_{csm,t}$, ksi (MPa)

$$= F_y + E_{sh}\left(\varepsilon_{csm,t} - \varepsilon_y\right) \tag{A-2-10}$$

$$\varepsilon_{csm,t} = \text{minimum}\left[15\varepsilon_y, 0.10\left(1 - F_y/F_u\right)\right] \tag{A-2-11}$$

User Note: Due to the high ductility and strain hardening in some stainless steels the rupture in the net section limit state may be associated with large deformation. Engineers requiring smaller deformation may examine the net section using a stress, f_{max}, smaller than F_u, as described in Section D2.

2.5. COMPRESSIVE STRENGTH

The design CSM compressive strength, $\phi_c P_n$, and the allowable CSM compressive strength, P_n/Ω_c, shall be determined as follows:

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

The nominal CSM compressive strength for the limit state of yielding or local buckling, P_n, is given by:

(a) When $\varepsilon_{csm}/\varepsilon_y < 1.0$

$$P_n = \frac{\varepsilon_{csm}}{\varepsilon_y} F_y A_g \tag{A-2-12}$$

(b) When $\varepsilon_{csm}/\varepsilon_y \geq 1.0$

$$P_n = f_{csm} A_g \tag{A-2-13}$$

where

f_{csm} = stress corresponding to ε_{csm}, ksi (MPa)

$$= F_y + E_{sh}\,\varepsilon_y \left(\frac{\varepsilon_{csm}}{\varepsilon_y} - 1\right) \tag{A-2-14}$$

2.6. FLEXURAL STRENGTH

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

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

The nominal CSM flexural strength for the limit state of yielding or local buckling, M_n, is given by:

(a) For I-shapes, channels, tees, HSS, and box-section members bent about an axis of symmetry

(1) When $\varepsilon_{csm}/\varepsilon_y < 1.0$

$$M_n = \frac{\varepsilon_{csm}}{\varepsilon_y} M_y \tag{A-2-15}$$

(2) When $\varepsilon_{csm}/\varepsilon_y \geq 1.0$

$$M_n = M_p \left(1 + \frac{E_{sh}}{E}\frac{S}{Z}\left(\frac{\varepsilon_{csm}}{\varepsilon_y} - 1\right) - \left(1 - \frac{S}{Z}\right)\Big/\left(\frac{\varepsilon_{csm}}{\varepsilon_y}\right)^{\alpha}\right) \tag{A-2-16}$$

where

$M_p = F_y Z$

$M_y = F_y S$

S = elastic section modulus about axis of bending, in.^{3} (mm^3)

Z = plastic section modulus about axis of bending, in.^{3} (mm^3)

α = bending coefficient, determined from Table A-2.6.1.

(b) For channels and angles bent about an axis that is not one of symmetry

The nominal CSM flexural strength of channels and angles bent about an axis that is not one of symmetry shall be determined as follows.

(1) The maximum attainable compressive strain, $\varepsilon_{csm,c}$, shall be determined in accordance with Section 2.3.1. The corresponding outer-fiber tensile strain, $\varepsilon_{csm,t}$, may then be determined assuming a linearly-varying through-depth strain distribution. Initially, $\varepsilon_{csm,t}$ may be calculated based on the location of the elastic neutral axis (ENA). The maximum design strain, $\varepsilon_{csm,max}$, shall then be taken as the maximum of $\varepsilon_{csm,c}$ and $\varepsilon_{csm,t}$.

(2) If $\varepsilon_{csm,max}$ is less than the yield strain, ε_y, the nominal CSM flexural strength shall be determined in accordance with Equation A-2-15, with $\varepsilon_{csm} = \varepsilon_{csm,max}$.

(3) If $\varepsilon_{csm,max}$ is greater than the yield strain, ε_y, the location of the design neutral axis shall be recalculated based on cross-section equilibrium or, as an approximation, it may be considered to lay at mid-distance between the elastic and plastic neutral axes. $\varepsilon_{csm,t}$ and $\varepsilon_{csm,max}$ shall then be recalculated using the new location of the neutral axis, and the nominal CSM flexural strength shall be determined in accordance with Equation A-2-16, with $\varepsilon_{csm} = \varepsilon_{csm,max}$ and using the values of the bending coefficient, α, taken from Table A-2.6.1.

As an alternative to using Equation A-2-16, the CSM flexural strength is permitted to be obtained by integration of stresses.

2.7. COMBINED FLEXURE AND COMPRESSION

The interaction of single-axis flexure and compression in I-shapes, square and rectangular HSS, and box-section members constrained to bend about a geometric axis (x or y), and of round HSS members shall be limited as follows:

(a) For I-shapes; square, rectangular, and round HSS; and box-section members subject to flexure and compression

(1) When $\lambda_l \leq 0.60$

Equations H1-1a and H1-1b shall be satisfied, but with P_c and M_c as defined in this section.

(2) When $\lambda_l > 0.60$

$$\frac{P_r}{P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \right) \leq 1.0 \qquad \text{(A-2-17)}$$

TABLE A-2.6.1
CSM Bending Coefficient, α

Member Type	Axis of Bending	Aspect Ratio	α
Square and rectangular HSS and box sections	Any	Any	2.0
Round HSS	Any	–	2.0
I-shaped members	Major	Any	2.0
	Minor	Any	1.2
Channels	Major	Any	2.0
	Minor	$h/b < 2$	1.5
		$h/b \geq 2$	1.0
Tees	Major	$h/b \leq 1$	1.0
		$h/b > 1$	1.5
	Minor	Any	1.2
Equal-leg angles	Any	–	1.0
Unequal-leg angles	Major	Any	1.5
	Minor	Any	1.0

(b) For round HSS subject to flexure and compression

(1) When $\lambda_l \leq 0.27$, Equations H1-1a and H1-1b shall be satisfied, but with P_c and M_c as defined in this section.

(2) When $\lambda_l > 0.27$, Equation A-2-17 shall be satisfied.

where

P_r = required compressive strength, determined in accordance with Chapter C, using LRFD or ASD load combinations, kips (N)

P_c = available CSM compressive strength, $\phi_c P_n$ or P_n/Ω_c, determined in accordance with Section 2.5, 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 CSM flexural strength, $\phi_b M_n$, or M_n/Ω_b, determined in accordance with Section 2.6, kip-in. (N-mm)

APPENDIX 3
FATIGUE

This appendix applies to austenitic and duplex stainless steel 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.

The appendix is organized as follows:

3.1. General Provisions
3.2. Calculation of Maximum Stresses and Stress Ranges
3.3. Plain Material and Welded Joints
3.4. Bolts and Threaded Parts
3.5. Fabrication and Erection Requirements for Fatigue
3.6. Nondestructive Examination Requirements for Fatigue

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 (EOR) 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

1. Allowable Stress Range

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:

$$F_{SR} = 1{,}000\left(\frac{C_f}{n_{SR}}\right)^{0.333} \geq F_{TH} \quad \text{(A-3-1)}$$

$$F_{SR} = 6\,900\left(\frac{C_f}{n_{SR}}\right)^{0.333} \geq F_{TH} \quad \text{(A-3-1M)}$$

where

C_f = constant from Table A-3.1 for the fatigue category

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:

$$F_{SR} = 100\left(\frac{1.5}{n_{SR}}\right)^{0.167} \geq 8 \text{ ksi} \quad \text{(A-3-2)}$$

$$F_{SR} = 690\left(\frac{1.5}{n_{SR}}\right)^{0.167} \geq 55 \text{ MPa} \quad \text{(A-3-2M)}$$

(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:

$$R_{PJP} = \frac{0.65 - 0.59\left(\frac{2a}{t_p}\right) + 0.72\left(\frac{w}{t_p}\right)}{t_p^{0.167}} \le 1.0 \tag{A-3-4}$$

$$R_{PJP} = \frac{1.12 - 1.01\left(\frac{2a}{t_p}\right) + 1.24\left(\frac{w}{t_p}\right)}{t_p^{0.167}} \le 1.0 \tag{A-3-4M}$$

$2a$ = length of the nonwelded root face in the direction of the thickness of the tension-loaded plate, in. (mm)

t_p = design thickness of tension loaded plate, as defined in Section B4.2, 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

$$= \frac{0.06 + 0.72\left(w / t_p\right)}{t_p^{0.167}} \leq 1.0 \tag{A-3-6}$$

$$= \frac{0.103 + 1.24\left(w / t_p\right)}{t_p^{0.167}} \leq 1.0 \tag{A-3-6M}$$

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.

2. Specific Requirements for Welded Connections in Cyclically Loaded Structures

In accordance with AWS D1.6/D1.6M, clause 4.14 and Figure 4.2, when a member is built up of two or more pieces, the pieces shall be connected along their longitudinal joints by sufficient continuous welds to make the pieces act in unison.

The following types of welds and joints are prohibited:

(a) In butt joints, PJP welds subject to tension normal to their longitudinal axes. In other joints, transversely loaded PJP welds are prohibited, unless fatigue design criteria allow for their application.

(b) Intermittent groove welds.

(c) Intermittent fillet welds.

(d) Plug and slot welds on primary tension members.

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 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 Section 8.4 (stress category G). The net area in tension, A_t, is given by Equation A-3-7 or A-3-7M:

$$A_t = \frac{\pi}{4}\left(d_b - \frac{0.9743}{n}\right)^2 \tag{A-3-7}$$

$$A_t = \frac{\pi}{4}\left(d_b - 0.9382\,p\right)^2 \tag{A-3-7M}$$

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 a pretension given by Equation J3-5, all axial load and moment applied to the joint plus effects of any prying action shall be assumed to be carried exclusively by the bolts or rods.

For joints in which the material within the grip is limited to steel and which are tensioned to a pretension given by Equation J3-5, 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.

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 ¼ 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 by predrilling or subpunching and reaming a hole, or by thermal cutting to form the radius of the cut.

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 in AWS D1.6/D1.6M, clause 8.12.2 or clause 8.13.2.

TABLE A-3.1
Fatigue Design Parameters

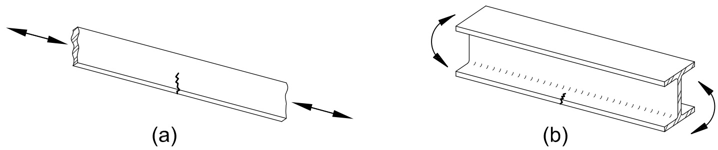
Description	Stress Category	Constant, C_f	Threshold, F_{TH}, ksi (MPa)	Potential Crack Initiation Point
SECTION 1—PLAIN MATERIAL AWAY FROM ANY WELDING				
1.1 Base metal with as-rolled or cleaned surfaces; cut edges with surface roughness value of 1,000 μin. (25 μm) or less, but without reentrant corners	A	25	24 (165)	Away from all welds or structural connections
1.2 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 radius, R, formed by predrilling, subpunching, and reaming, or cut and ground to a bright metal surface	C	4.4	10 (69)	
$R \geq 3/8$ in. (10 mm) and the radius, R, need not be ground to a bright metal surface	E′	0.39	2.6 (18)	
1.3 Rolled cross sections with weld access holes made to requirements of Section J1.6				At reentrant corner of weld access hole
Access hole $R \geq 1$ in. (25 mm) with radius, R, formed by predrilling, subpunching, and reaming, or cut and ground to a bright metal surface	C	4.4	10 (69)	
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)	
1.4 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 bolts in joints satisfying all requirements for slip-critical connections	B	12	16 (110)	Through gross section near hole

TABLE A-3.1 (continued)
Fatigue Design Parameters

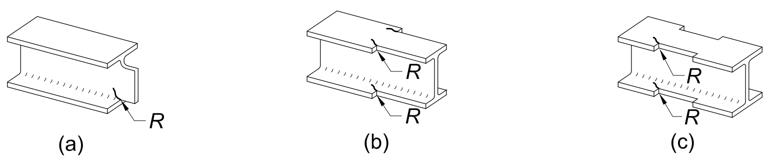
Illustrative Typical Examples

SECTION 1—PLAIN MATERIAL AWAY FROM ANY WELDING

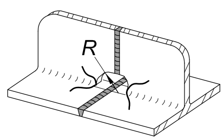
1.1



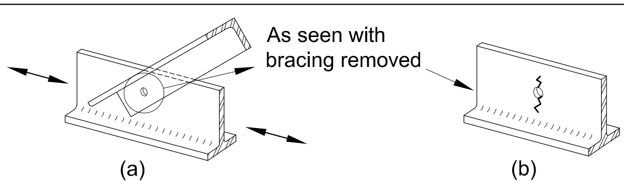
1.2



1.3

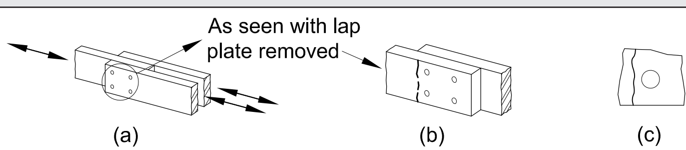


1.4



SECTION 2—CONNECTED MATERIAL IN MECHANICALLY FASTENED JOINTS

2.1



(Note: Figures are for slip-critical bolted connections.)

TABLE A-3.1
Fatigue Design Parameters

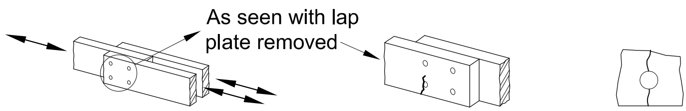
Description	Stress Category	Constant, C_f	Threshold, F_{TH}, ksi (MPa)	Potential Crack Initiation Point
SECTION 2—CONNECTED MATERIAL IN MECHANICALLY FASTENED JOINTS (cont'd)				
2.2 Base metal at net section of bolted joints, designed on the basis of bearing resistance, but fabricated and installed to all requirements for slip-critical connections	B	12	16 (110)	In net section originating at side of hole
2.3 Base metal at net section of pin plate	E	1.1	4.5 (31)	In net section originating at side of hole
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 $R \geq \frac{3}{8}$ in. (10 mm) and the radius, R, need not be ground to a bright metal surface	E′	0.39	2.6 (18)	

TABLE A-3.1 (continued)
Fatigue Design Parameters

Illustrative Typical Examples

SECTION 2—CONNECTED MATERIAL IN MECHANICALLY FASTENED JOINTS (cont'd)

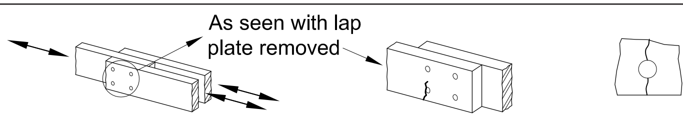
2.2



(a) (b) (c)

(Note: Figures are for bolted connections designed to bear, meeting the requirements of slip-critical connections.)

2.3

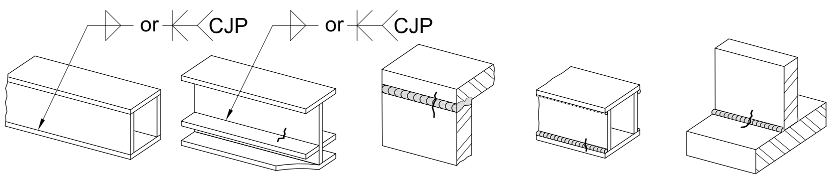


(a) (b) (c)

(Note: Figures are for snug-tightened bolts, rivets, or other mechanical fasteners.)

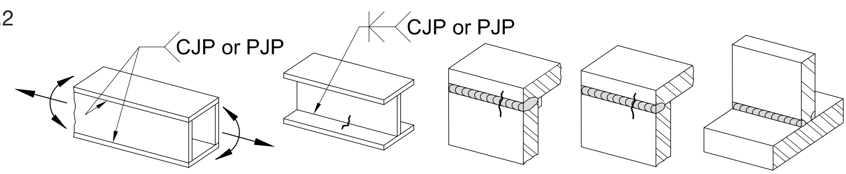
SECTION 3—WELDED JOINTS JOINING COMPONENTS OF BUILT-UP MEMBERS

3.1



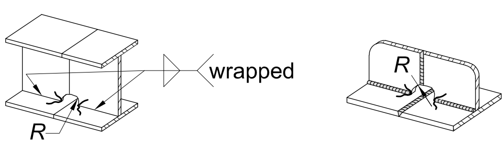
(a) (b) (c) (d) (e)

3.2



(a) (b) (c) (d) (e)

3.3



(a) (b)

TABLE A-3.1 (continued)
Fatigue Design Parameters

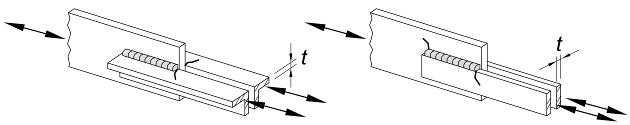
Description	Stress Category	Constant, C_f	Threshold, F_{TH}, ksi (MPa)	Potential Crack Initiation Point
SECTION 4—LONGITUDINAL FILLET WELDED END CONNECTIONS				
4.1 Base metal at junction of axially loaded members with longitudinally welded end connections; welds are on each side of the axis of the member to balance weld stresses				Initiating from end of any weld termination extending into the base metal
$t \le 0.5$ in. (13 mm)	E	1.1	4.5 (31)	
$t > 0.5$ in. (13 mm)	E′	0.39	2.6 (18)	
where t = connected member thickness, as shown in Case 4.1 figure, in. (mm)				
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 1:2½ and inspected in accordance with Section 3.6	B	12	16 (110)	From internal discontinuities in metal or along the fusion boundary
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

TABLE A-3.1 (continued)
Fatigue Design Parameters

Illustrative Typical Examples

SECTION 4—LONGITUDINAL FILLET WELDED END CONNECTIONS

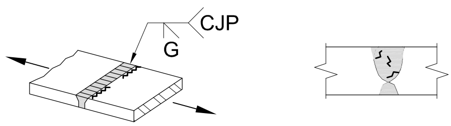
4.1



(a) (b)

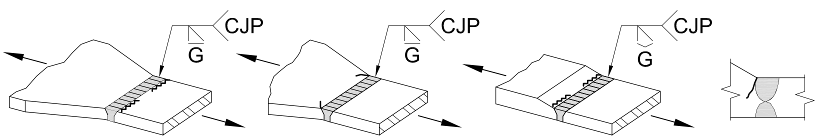
SECTION 5—WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS

5.1



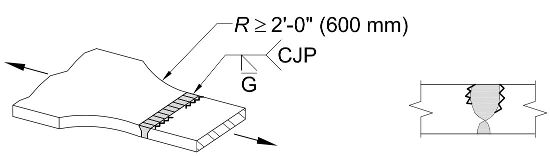
(a) (b)

5.2



(a) (b) (c) (d)

5.3



(a) (b)

TABLE A-3.1 (continued)
Fatigue Design Parameters

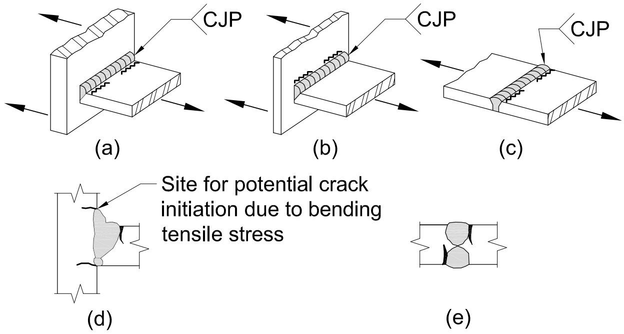
Description	Stress Category	Constant, C_f	Threshold, F_{TH}, ksi (MPa)	Potential Crack Initiation Point
SECTION 5—WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS (cont'd)				
5.4 Weld metal and base metal in or adjacent to CJP groove welds in T- or corner-joints or splices, without transitions in thickness or with transition in thickness having slopes no greater than 1:2½, 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 ½ 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

Illustrative Typical Examples

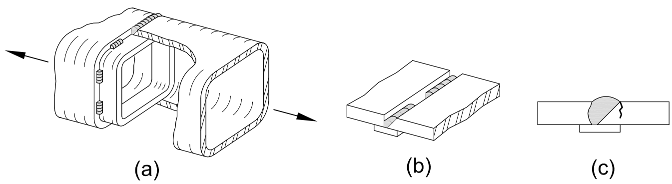
SECTION 5—WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS (cont'd)

5.4

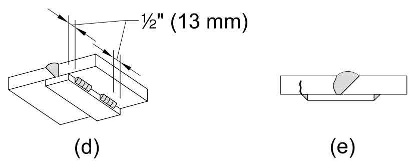


5.5

Category D

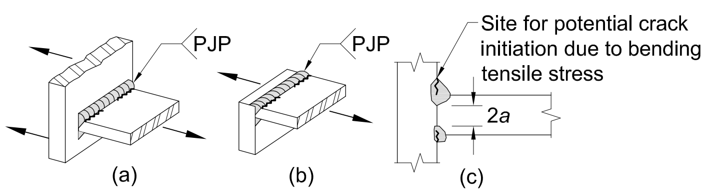


Category E



5.6

Toe Crack
Category C



Root Crack
Category C'

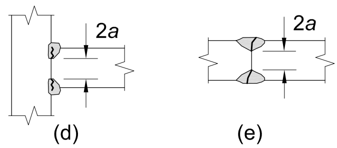


TABLE A-3.1 (continued)
Fatigue Design Parameters

Description	Stress Category	Constant, C_f	Threshold, F_{TH}, ksi (MPa)	Potential Crack Initiation Point
SECTION 5—WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS (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. $\leq R <$ 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)	

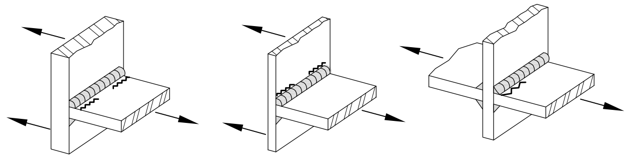
TABLE A-3.1 (continued)
Fatigue Design Parameters

Illustrative Typical Examples

SECTION 5—WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS (cont'd)

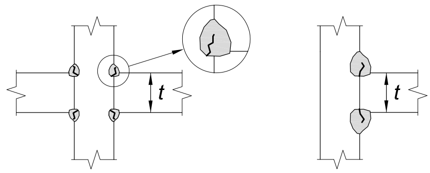
5.7

Toe Crack
Category C



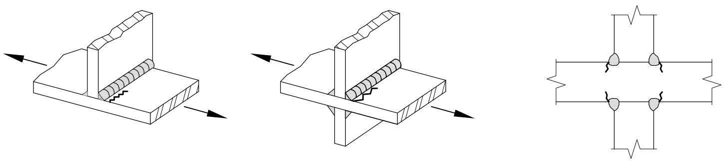
(a) (b) (c)

Root Crack
Category C''



(d) (e)

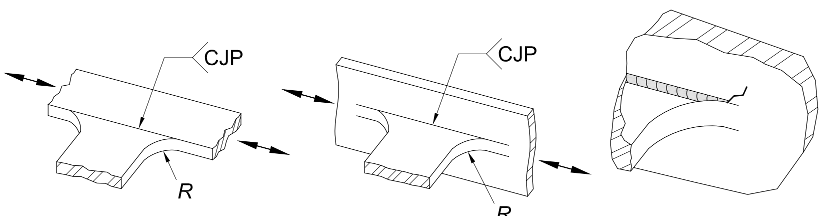
5.8



(a) (b) (c)

SECTION 6—BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS

6.1



(a) (b) (c)

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 (cont'd)				
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)	

TABLE A-3.1 (continued)
Fatigue Design Parameters

Illustrative Typical Examples

SECTION 6—BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)

6.2

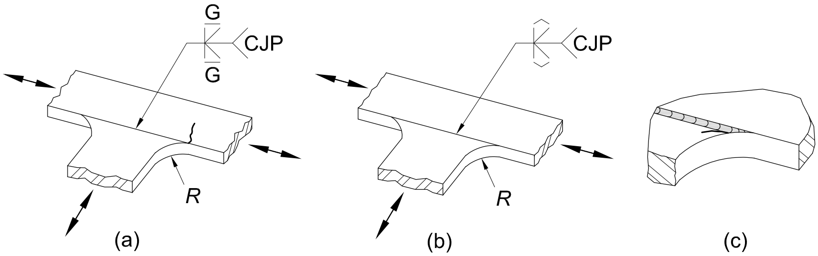


TABLE A-3.1 (continued)
Fatigue Design Parameters

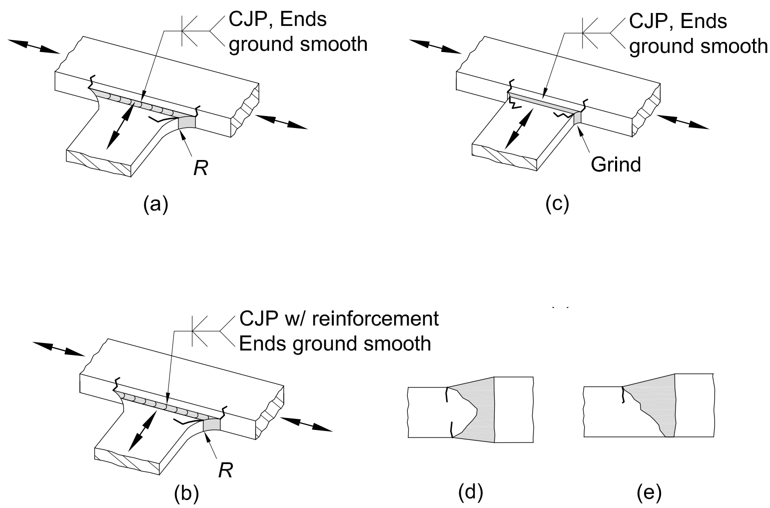
Description	Stress Category	Constant, C_f	Threshold, F_{TH}, ksi (MPa)	Potential Crack Initiation Point
SECTION 6—BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)				
6.3 Base metal at details of unequal thickness attached by CJP groove welds, subject to transverse loading, with or without 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
$R \leq 2$ 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
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)	
$R \leq 2$ in. (50 mm)	E	1.1	4.5 (31)	

TABLE A-3.1 (continued)
Fatigue Design Parameters

Illustrative Typical Examples

SECTION 6—BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)

6.3



6.4

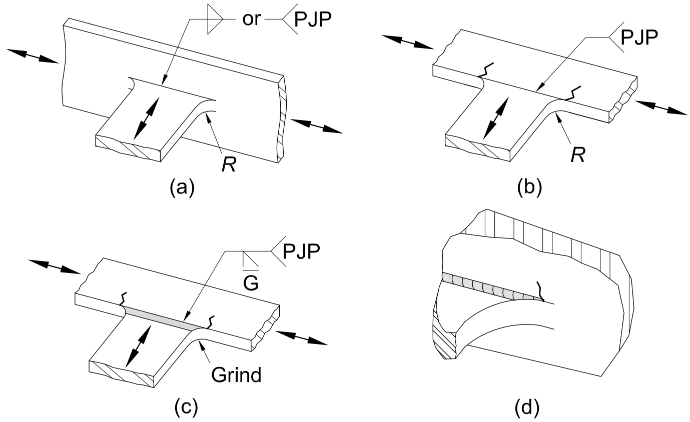


TABLE A-3.1 (continued)
Fatigue Design Parameters

Description	Stress Category	Constant, C_f	Threshold, F_{TH}, ksi (MPa)	Potential Crack Initiation Point
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 ≤ lesser of 12b or 4 in. (100 mm)	D	2.2	7 (48)	
a > lesser of 12b or 4 in. (100 mm) when b ≤ 0.8 in. (20 mm)	E	1.1	4.5 (31)	
a > 4 in. (100 mm) when 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)	

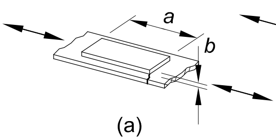
[a] "Attachment," as used herein, is defined as any steel detail welded to a member that causes a deviation in the stress flow in the member and, thus, reduces the fatigue resistance. The reduction is due to the presence of the attachment, not due to the loading on the attachment.

TABLE A-3.1 (continued)
Fatigue Design Parameters

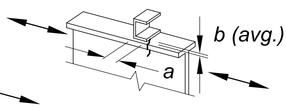
Illustrative Typical Examples

SECTION 7—BASE METAL AT SHORT ATTACHMENTS[a]

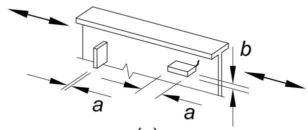
7.1



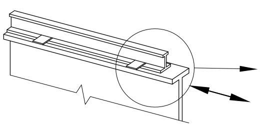
(a)



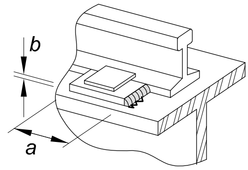
(b)



(c)

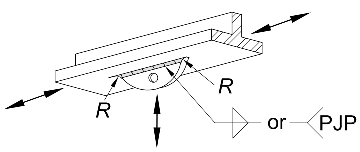


(d)

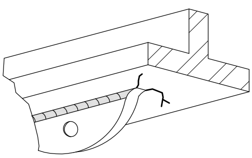


(e)

7.2



(a)



(b)

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 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
8.2 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
8.3 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
8.4 Bolts, threaded anchor rods, and hanger rods, whether pretensioned in accordance with Equation J3-5, 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

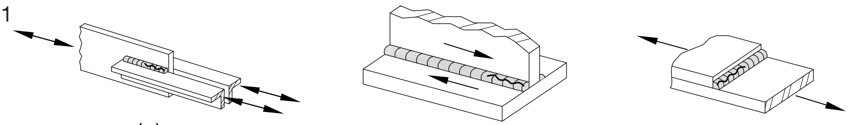
[a] "Attachment," as used herein, is defined as any steel detail welded to a member that causes a deviation in the stress flow in the member and, thus, reduces the fatigue resistance. The reduction is due to the presence of the attachment, not due to the loading on the attachment.

TABLE A-3.1 (continued)
Fatigue Design Parameters

Illustrative Typical Examples

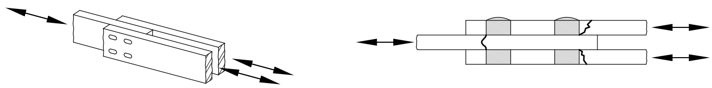
SECTION 8—MISCELLANEOUS

8.1



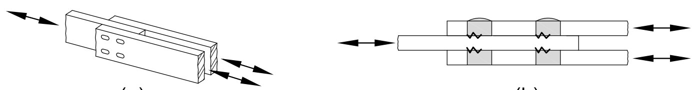
(a) (b) (c)

8.2



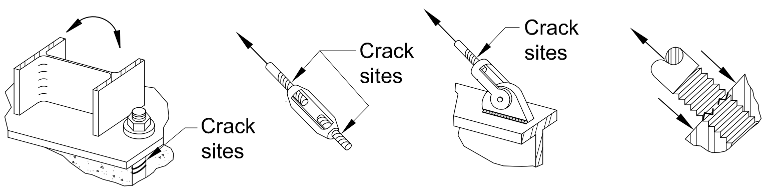
(a) (b)

8.3



(a) (b)

8.4



(a) (b) (c) (d)

APPENDIX 4

STRUCTURAL DESIGN FOR FIRE CONDITIONS

This appendix provides criteria for the design and evaluation of austenitic and duplex stainless 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 austenitic and duplex stainless steels at elevated temperatures that cause progressive decrease in strength and stiffness of structural components and systems at elevated temperatures.

User Note: Throughout this appendix, 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 subject 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 Section 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.

A notional load, $N_i = 0.002Y_i$, as defined in Section C2.2b, where N_i = notional load applied at framing level i and Y_i = gravity load from Equation A-4-1 acting on framing level i, shall be applied in combination with the loads stipulated in Equation A-4-1. Unless otherwise stipulated by the applicable building code, D, L, and S shall be the nominal loads specified in ASCE/SEI 7.

> **User Note:** The effect of initial imperfections may be taken into account by direct modeling of imperfections in the analysis. In typical building structures, when evaluating frame stability, the important imperfection is the out-of-plumbness of columns.

5. Avoidance of Embrittlement Due to Contact with Molten Zinc

Precautions shall be taken to ensure that in the event of a fire, molten zinc from galvanized steel cannot drip or run onto the stainless steel and cause embrittlement.

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 Strengths at Elevated Temperatures

Material properties at elevated temperatures shall be determined from test data. In the absence of such data, it is permitted to use the material properties stipulated in this section for austenitic stainless steels S30400/S30403 and S31600/S31603 and duplex stainless steels S32003, S32101, S32202, S32205, S32304, S82011, and S82441.

User Note: Refer to the Commentary for guidance on the mechanical properties of precipitation hardening at high temperatures.

3a. Thermal Elongation

The coefficients of expansion for calculations at temperatures ranging from 68 to 1,800°F (20 to 980°C) shall be taken as follows:

Austenitic stainless steels: 11×10^{-6}/°F (20×10^{-6}/°C).

Duplex stainless steels: 9×10^{-6}/°F (16×10^{-6}/°C).

3b. Mechanical Properties at Elevated Temperatures

The deterioration in strength and stiffness of structural members, components, and systems shall be taken into account in the structural analysis of the frame. For stainless steel, the values $F_y(T)$, $F_u(T)$, $F_2(T)$, and $E(T)$ at elevated temperature to be used in structural analysis, expressed as the ratio with respect to the property at ambient, which is assumed to be 68°F (20°C), shall be defined as in Tables A-4.2.1 through A-4.2.4. It is permitted to linearly interpolate between the tabulated values.

$F_2(T)$ is the stress at 2% strain at elevated temperatures, ksi (MPa), and is used in the material model given in Appendix 7, Section 7.2.

TABLE A-4.2.1 Properties of Austenitic Type S30400/S30403 at Elevated Temperatures

Steel Temperature, °F (°C)	$k_E = E(T)/E$	$k_y = F_y(T)/F_y$	$k_2 = F_2(T)/F_y$	$k_u = F_u(T)/F_u$
68 (20)	1.00	1.00	1.31	1.00
200 (93)	0.96	0.80	1.05	0.83
400 (200)	0.92	0.65	0.88	0.72
600 (320)	0.87	0.59	0.81	0.68
750 (400)	0.84	0.55	0.78	0.66
800 (430)	0.83	0.54	0.77	0.65
1000 (540)	0.78	0.48	0.71	0.58
1200 (650)	0.74	0.42	0.61	0.47
1400 (760)	0.66	0.30	0.43	0.31
1600 (870)	0.50	0.18	0.23	0.16
1800 (980)	0.24	0.08	0.10	0.09
2000 (1100)	0.11	0.05	0.06	0.05

TABLE A-4.2.2 Properties of Austenitic Type S31600/S31603 and S32100 at Elevated Temperatures

Steel Temperature, °F (°C)	$k_E = E(T)/E$	$k_y = F_y(T)/F_y$	$k_2 = F_2(T)/F_y$	$k_u = F_u(T)/F_u$
68 (20)	1.00	1.00	1.19	1.00
200 (93)	0.96	0.87	1.14	0.88
400 (200)	0.92	0.72	0.98	0.80
600 (320)	0.87	0.66	0.91	0.78
750 (400)	0.84	0.62	0.85	0.77
800 (430)	0.83	0.61	0.84	0.76
1000 (540)	0.78	0.58	0.79	0.71
1200 (650)	0.74	0.53	0.72	0.59
1400 (760)	0.66	0.45	0.57	0.41
1600 (870)	0.50	0.27	0.33	0.23
1800 (980)	0.24	0.15	–	0.12
2000 (1100)	0.11	0.07	–	0.07

TABLE A-4.2.3
Properties of Duplex Types S32202 and S32304 at Elevated Temperatures

Steel Temperature, °F (°C)	$k_E = E(T)/E$	$k_y = F_y(T)/F_y$	$k_2 = F_2(T)/F_y$	$k_u = F_u(T)/F_u$
68 (20)	1.00	1.00	1.15	1.00
200 (93)	0.96	0.84	0.96	0.95
400 (200)	0.92	0.75	0.82	0.87
600 (320)	0.87	0.67	0.76	0.78
750 (400)	0.84	0.58	0.70	0.70
800 (430)	0.83	0.54	0.67	0.67
1000 (540)	0.78	0.37	0.53	0.54
1200 (650)	0.74	0.21	0.37	0.40
1400 (760)	0.66	0.10	0.20	0.25
1600 (870)	0.50	0.05	0.08	0.12
1800 (980)	0.24	–	–	–
2000 (1100)	0.11	–	–	–

TABLE A-4.2.4
Properties of Duplex Types S32003, S32101, S32205, S82011, and S82441 at Elevated Temperatures

Steel Temperature, °F (°C)	$k_E = E(T)/E$	$k_y = F_y(T)/F_y$	$k_2 = F_2(T)/F_y$	$k_u = F_u(T)/F_u$
68 (20)	1.00	1.00	1.12	1.00
200 (93)	0.96	0.84	0.97	0.96
400 (200)	0.92	0.70	0.86	0.91
600 (320)	0.87	0.64	0.81	0.87
750 (400)	0.84	0.60	0.76	0.82
800 (430)	0.83	0.58	0.73	0.79
1000 (540)	0.78	0.49	0.62	0.65
1200 (650)	0.74	0.35	0.46	0.47
1400 (760)	0.66	0.20	0.27	0.28
1600 (870)	0.50	0.09	0.14	0.16
1800 (980)	0.24	0.02	0.05	0.07
2000 (1100)	0.11	–	–	–

4. Structural Design Requirements

4a. General Structural Integrity

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.

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 develop the strength of the connected members or the forces indicated above. 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.

4c. Design by Advanced Methods of Analysis

Design by advanced methods of analysis is permitted for the design of all structural stainless 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 subject 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. Boundary conditions and connection fixity must represent the proposed structural design. Material properties shall be defined as per Section 4.2.3.

> **User Note:** The material model at elevated temperatures included in Appendix 7 can be used to represent the mechanical response of austenitic and duplex stainless steel structural members or a structural system under fire conditions.

The resulting analysis shall address all relevant limit states, such as excessive deflections, connection ruptures, and overall 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 fire.

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.

It is permitted to model the thermal response of structural stainless steel members using a one-dimensional heat transfer equation with heat input as determined by the design-basis fire defined in Section 4.2.1, using the temperature equal to the maximum structural stainless steel temperature. For flexural members, the maximum structural stainless steel temperature shall be assigned to the bottom flange.

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 (e).

(a) Design for tension
Nominal strength for tension shall be determined using the provisions of Chapter D, with steel properties as stipulated in Section 4.2.3b and assuming a uniform temperature over the cross section using the temperature equal to the maximum structural stainless steel temperature.

(b) Design for compression
The nominal compressive strength for the limit state of yielding and flexural buckling of members with nonslender elements as defined in Section B4.1 for elements in axial compression shall be determined using the provisions of Chapter E with stainless steel properties as stipulated in Section 4.2.3b and Equations A-4-2 and A-4-3 used in lieu of Equations E3-2 through E3-4 to calculate the nominal compressive strength for flexural buckling.

(1) When $\dfrac{L_c}{r} \le 5.62\sqrt{\dfrac{E(T)}{F_y(T)}} \quad \left[\dfrac{F_y(T)}{F_e(T)} \le 3.20\right]$

$$F_{cr}(T) = 1.2\left\{\left[\frac{\beta_2(T)}{3.84}\right]^{\left[\frac{F_y(T)}{3.2F_e(T)}\right]^{\alpha(T)}}\right\} F_y(T) \le F_y(T) \tag{A-4-2}$$

(2) When $\frac{L_c}{r} > 5.62\sqrt{\frac{E(T)}{F_y(T)}}$ $\left[\text{or } \frac{F_y(T)}{F_e(T)} > 3.20\right]$

$$F_{cr}(T) = \beta_2(T) F_e(T) \tag{A-4-3}$$

where

$E(T)$ = modulus of elasticity at elevated temperature determined using the coefficients from Tables A-4.2.1, A-4.2.2, A-4.2.3, or A-4.2.4

$F_y(T)$ = yield stress at elevated temperature determined using the coefficients from Tables A-4.2.1, A-4.2.2, A-4.2.3, or A-4.2.4

$F_e(T)$ = critical elastic buckling stress calculated from Equation E3-5 with the elastic modulus, $E(T)$, at elevated temperature

$\alpha(T)$, $\beta_2(T)$ = flexural buckling coefficients at elevated temperature determined from Table A-4.2.5

(c) Design for flexure

For structural stainless steel beams, it is permitted to assume that the calculated bottom flange temperature is constant over the depth of the member.

The nominal flexural strength for the limit states of yielding and lateral-torsional buckling of doubly symmetric I-shaped members and channels, having compact webs and compact flanges as defined in Section B4.1 for flexure shall be determined using the provisions of Chapter F with stainless steel properties as stipulated in Section 4.2.3b and using the lateral-torsional buckling coefficients at elevated temperature given in Table A-4.2.6 in lieu of the lateral-torsional buckling coefficients given in Table F2.1.

The material properties at elevated temperatures, $E(T)$ and $F_y(T)$, and the k_E and k_y coefficients are calculated in accordance with Tables A-4.2.1 to A-4.2.4, and other terms are as defined in Chapter F.

(d) Design for shear

Nominal strength for shear shall be determined in accordance with the provisions of Chapter G, with stainless steel properties as stipulated in Section 4.2.3b and assuming a uniform temperature over the cross section.

(e) Design for combined forces

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 (c).

Table A-4.2.5
Flexural Buckling Coefficients at Elevated Temperatures

Member Type	Alloy Family	$\alpha(T)$	$\beta_2(T)$
Rolled or built-up I-shaped sections buckling about the minor axis, and other sections not specified in this table	Austenitic and duplex	0.56	$0.69-0.25\left(1-\frac{k_y}{k_E}\right)$
Rolled or built-up I-shaped sections buckling about the major axis, welded box sections, and round HSS	Austenitic and duplex	0.58	$0.82-0.40\left(1-\frac{k_y}{k_E}\right)$
Rectangular HSS	Austenitic and duplex	$0.69-0.30\left(1-\frac{k_y}{k_E}\right)$	0.82

Table A-4.2.6
Lateral-Torsional Buckling Coefficients at Elevated Temperatures

Alloy Family	$\beta_{LT}(T)$	$\beta_{p,LT}(T)$	$\beta_{y,LT}(T)$
Austenitic	$0.82-0.10\left(1-\frac{k_y}{k_E}\right)$	$0.90-0.11\left(1-\frac{k_y}{k_E}\right)$	0.40
Duplex	$0.86-0.10\left(1-\frac{k_y}{k_E}\right)$	$1.10-0.13\left(1-\frac{k_y}{k_E}\right)$	0.50

4.3. DESIGN BY QUALIFICATION TESTING

1. Qualification Standards

Structural members and components in structural stainless steel buildings shall be qualified for the rating period in conformance with ASTM E119.

2. Restrained Construction

For floor and roof assemblies and individual beams in buildings, a restrained condition exists when the surrounding or supporting structure is capable of resisting forces and accommodating deformations caused by thermal expansion throughout the range of anticipated elevated temperatures.

Structural stainless steel beams, girders, and frames supporting concrete slabs that are welded or bolted to integral framing members shall be considered restrained construction.

3. Unrestrained Construction

Structural stainless 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 structural stainless steel member bearing on a wall in a single span or at the end span of multiple spans shall be considered unrestrained unless the wall has been designed and detailed to resist effects of thermal expansion.

APPENDIX 5
EVALUATION OF EXISTING STRUCTURES

This appendix applies to the evaluation of the strength and stiffness under static loads of existing structures by structural analysis, by load tests, or by a combination of structural analysis and load tests when specified by the EOR or in the contract documents. For such evaluation, the stainless steel materials are not limited to those listed in Section A3.1. This appendix does not address load testing for the effects of seismic loads or moving loads (vibrations). Section 5.4 is only applicable to static vertical gravity loads applied to existing roofs or floors.

The appendix is organized as follows:

5.1. General Provisions
5.2. Material Properties
5.3. Evaluation by Structural Analysis
5.4. Evaluation by Load Tests
5.5. Evaluation Report

5.1. GENERAL PROVISIONS

These provisions shall be applicable when the evaluation of an existing steel structure is specified for (a) verification of a specific set of design loadings or (b) determination of the available strength of a load-resisting member or system. The evaluation shall be performed by structural analysis (Section 5.3), by load tests (Section 5.4), or by a combination of structural analysis and load tests, when specified in the contract documents by the EOR. 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.

5.2. MATERIAL PROPERTIES

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. Where available, 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 considered in evaluation by structural analysis (Section 5.3) or load tests (Section 5.4). Such properties shall include the yield stress, tensile strength, and percent elongation. Where available, certified material test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM A480/A480M or A484/A484M, as applicable, is 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. Where available, results from certified material test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM procedures is 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 A3.1c are critical to the performance of the structure, the Charpy V-notch toughness shall be determined in accordance with the provisions of Section A3.1c. If the notch toughness so determined does not meet the provisions of Section A3.1c, 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 AWS D1.6/D1.6M, are not met, the EOR shall determine if remedial actions are required.

6. Bolts

Representative samples of bolts shall be inspected to determine markings and classifications. Where bolts cannot be properly identified, 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 have a specified minimum yield stress of 20 ksi (140 MPa) and a tensile strength of 65 ksi (450 MPa) is permitted.

5.3. EVALUATION BY STRUCTURAL ANALYSIS

1. Dimensional Data

All dimensions used in the evaluation, such as spans, column heights, member spacings, bracing locations, cross-section dimensions, thicknesses, and connection details, shall be determined from a field survey. Alternatively, when available, it 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 of this Specification.

3. Serviceability Evaluation

Where required, the deformations at service loads shall be calculated and reported.

5.4. EVALUATION BY LOAD TESTS

1. Determination of Load Rating by Testing

To determine the load rating of an existing floor or roof structure by testing, a test load shall be applied incrementally in accordance with the EOR's plan. The structure shall be visually inspected for signs of distress or imminent failure at each load level. Appropriate measures shall be taken if these or any other unusual conditions are encountered.

The tested strength of the structure shall be taken as the maximum applied test load plus the in-situ dead load. The live load rating of a floor structure shall be determined by setting the tested strength equal to $1.2D + 1.6L$, where D is the nominal dead load and L is the nominal live load rating for the structure. 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 shall be considered once the service load level is attained and after the onset of inelastic structural behavior is identified to document the amount of permanent set and the magnitude of the inelastic deformations. Deformations of the structure, such as member deflections, shall be monitored at critical locations during the test, referenced to the initial position before loading. It shall be demonstrated, while maintaining maximum test load for one hour, that the deformation of the structure does not increase by more than 10% above that at the beginning of the holding period. It is permissible to repeat the sequence if necessary to demonstrate compliance.

Deformations of the structure shall also be recorded 24 hours after the test loading is removed to determine the amount of permanent set. Because the amount of acceptable permanent deformation depends on the specific structure, no limit is specified for permanent deformation at maximum loading. Where it is not feasible to load test the entire structure, a segment or zone of not less than one complete bay representative of the most critical conditions shall be selected.

User Note: The characteristically low proportional limit exhibited by stainless steel may lead to greater permanent deformations than experienced with other steel alloys; a stainless steel member is therefore less likely to return to its original undeformed condition upon removal of the load.

2. 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 1 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. Finally, the report shall indicate whether the structure, including all members and connections, is adequate to withstand the load effects.

APPENDIX 6

MEMBER STABILITY BRACING

This appendix addresses the minimum strength and stiffness necessary to provide a brace that allows the member to develop the required strength in an austenitic or duplex stainless steel 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 on the 2016 AISC *Specification for Structural Steel Buildings*.

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. 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 on the 2016 AISC *Specification for Structural Steel Buildings*.

A panel brace (formerly referred to as a relative brace) controls the angular deviation of a segment of the braced member between braced points (i.e., 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 on the 2016 AISC *Specification for Structural Steel Buildings*, Appendix 6, 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 on the 2016 AISC *Specification for Structural Steel Buildings* 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:

$$V_{br} = 0.0075P_r \tag{A-6-1}$$

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:

$$P_{br} = 0.015P_r \tag{A-6-3}$$

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 subject 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:

$$V_{br} = 0.015\left(\frac{M_r C_d}{h_o}\right) \tag{A-6-5}$$

and, the required shear stiffness of the bracing system is:

$$\beta_{br} = \frac{1}{\phi}\left(\frac{4M_r C_d}{L_{br} h_o}\right) \quad \text{(LRFD)} \tag{A-6-6a}$$

$$\beta_{br} = \Omega\left(\frac{4M_r C_d}{L_{br} h_o}\right) \quad \text{(ASD)} \tag{A-6-6b}$$

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

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:

$$P_{br} = 0.03\left(\frac{M_r C_d}{h_o}\right) \tag{A-6-7}$$

and, the required stiffness of the brace is:

$$\beta_{br} = \frac{1}{\phi}\left(\frac{10M_r C_d}{L_{br} h_o}\right) \quad \text{(LRFD)} \tag{A-6-8a}$$

$$\beta_{br} = \Omega\left(\frac{10M_r C_d}{L_{br} h_o}\right) \quad \text{(ASD)} \tag{A-6-8b}$$

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

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:

$$M_{br} = 0.03M_r \tag{A-6-9}$$

and the required flexural stiffness of the brace is:

$$\beta_{br} = \frac{\beta_T}{\left(1 - \dfrac{\beta_T}{\beta_{sec}}\right)} \tag{A-6-10}$$

where

$$\beta_T = \frac{1}{\phi}\frac{2.4L}{nEI_{yeff}}\left(\frac{M_r}{C_b}\right)^2 \text{ (LRFD)} \tag{A-6-11a}$$

$$= \Omega\frac{2.4L}{nEI_{yeff}}\left(\frac{M_r}{C_b}\right)^2 \text{ (ASD)} \tag{A-6-11b}$$

$$\beta_{sec} = \frac{3.3E}{h_o}\left(\frac{1.5h_o t_w^3}{12} + \frac{t_{st} b_s^3}{12}\right) \tag{A-6-12}$$

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 stainless steel, ksi (MPa)
= 28,000 ksi (193 000 MPa) for austenitic, and 29,000 ksi (200 000 MPa) for duplex stainless steel
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)

$\dfrac{M_r}{C_b}$ = 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 = design thickness of beam web, as defined in Section B4.2, in. (mm)

t_{st} = design thickness of web stiffener, as defined in Section B4.2, 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.

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 on the 2016 AISC *Specification for Structural Steel Buildings*.

APPENDIX 7
MODELING OF MATERIAL BEHAVIOR

This appendix provides analytical expressions for modeling of material behavior at ambient and at elevated temperatures, as well as expressions for calculating the secant and tangent modulus, of austenitic and duplex stainless steel structural members.

The appendix is organized as follows:

7.1 MATERIAL BEHAVIOR AT AMBIENT TEMPERATURE

1. Stress-Strain Behavior

The nonlinear stress-strain behavior of austenitic and duplex stainless steel, including strain hardening, can be determined in accordance with Equations A-7-1a and A-7-1b.

For $f \le F_y$

$$\varepsilon = \frac{f}{E} + 0.002\left(\frac{f}{F_y}\right)^n \tag{A-7-1a}$$

For $F_y < f \le F_u$

$$\varepsilon = 0.002 + \frac{F_y}{E} + \frac{f - F_y}{E_{ty}} + \left(\varepsilon_u - 0.002 - \frac{F_y}{E} - \frac{F_u - F_y}{E_{ty}}\right)\left(\frac{f - F_y}{F_u - F_y}\right)^m \tag{A-7-1b}$$

where

E = modulus of elasticity of stainless steel, ksi (MPa)
= 28,000 ksi (193 000 MPa) for austenitic, and 29,000 ksi (200 000 MPa) for duplex stainless steel

E_{ty} = tangent modulus at the specified minimum yield stress, determined in accordance with Equation A-7-4 by replacing *f* with F_y, ksi (MPa)

F_y = specified minimum yield stress, ksi (MPa)

F_u = specified minimum tensile strength, ksi (MPa)

n = strain hardening coefficient
= 7 for austenitic stainless steel
= 8 for duplex stainless steel

$$m = 1 + 2.8\frac{F_y}{F_u} \tag{A-7-2}$$

f = engineering stress, ksi (MPa)

ε = engineering strain

ε_u = strain at the ultimate tensile stress, which may be approximated in accordance with Equation A-7-3 if not known

$$= 1 - \frac{F_y}{F_u} \leq \varepsilon_f \tag{A-7-3}$$

ε_f = specified minimum elongation after rupture determined over a length of 2 in. (50 mm)

2. Tangent Modulus, E_t

The tangent modulus, E_t, of austenitic and duplex stainless steel can be determined in accordance with Equation A-7-4 for any stress level $f \leq F_y$:

$$E_t = \frac{EF_y}{F_y + 0.002nE\left(\frac{f}{F_y}\right)^{n-1}} \tag{A-7-4}$$

3. Secant Modulus, E_s

The secant modulus, E_s, of austenitic and duplex stainless steel can be determined in accordance with Equation A-7-5 for any stress level $f \leq F_y$:

$$E_s = \frac{E}{1 + 0.002\frac{E}{f}\left(\frac{f}{F_y}\right)^{n}} \tag{A-7-5}$$

User Note: The secant modulus may be required to check the deflection of any cross section made of stainless steel, as specified in Section L2.

7.2 MATERIAL BEHAVIOR AT ELEVATED TEMPERATURES

The nonlinear stress-strain behavior of austenitic and duplex stainless steel at elevated temperatures, including strain hardening, can be determined in accordance with Equations A-7-6a and A-7-6b. This material model shall be used to represent the deterioration of strength and stiffness exhibited by austenitic and duplex stainless steel structural members or a structural system at elevated temperatures when the structural design for fire conditions is carried out using advanced methods of analysis, as specified in Section 4.2.4c.

For $f(T) \leq F_y(T)$

$$\varepsilon(T) = \frac{f(T)}{E(T)} + 0.002\left[\frac{f(T)}{F_y(T)}\right]^{n(T)} \tag{A-7-6a}$$

For $F_y(T) < f(T) \le F_u(T)$

$$\varepsilon(T) = \varepsilon_y(T) + \frac{f(T) - F_y(T)}{E_{ty}(T)} + \left[\varepsilon_u(T) - \varepsilon_y(T) - \frac{F_u(T) - F_y(T)}{E_{ty}(T)}\right]\left[\frac{f(T) - F_y(T)}{F_u(T) - F_y(T)}\right]^{m(T)} \quad \text{(A-7-6b)}$$

where

$E(T)$ = modulus of elasticity of stainless steel at elevated temperatures, ksi (MPa)
= $k_E E$

$E_{ty}(T)$ = tangent modulus at the yield stress at elevated temperatures, ksi (MPa)

$$= \frac{E(T)}{1 + 0.002\,n(T)\dfrac{E(T)}{F_y(T)}} \quad \text{(A-7-7)}$$

$F_y(T)$ = yield stress at elevated temperatures, ksi (MPa)
= $k_y F_y$

$F_u(T)$ = tensile strength at elevated temperatures, ksi (MPa)
= $k_u F_u$

$F_2(T)$ = stress at 2% strain at elevated temperatures, ksi (MPa)
= $k_2 F_y$

$n(T)$ = strain hardening coefficient at elevated temperatures
= 7 for austenitic stainless steel
= 8 for duplex stainless steel

$$m(T) = \frac{\ln\left(\dfrac{0.02 - \varepsilon_y(T) - \left(\dfrac{F_2(T) - F_y(T)}{E_{ty}(T)}\right)}{\varepsilon_u(T) - \varepsilon_y(T) - \left(\dfrac{F_u(T) - F_y(T)}{E_{ty}(T)}\right)}\right)}{\ln\left(\dfrac{F_2(T) - F_y(T)}{F_u(T) - F_y(T)}\right)}, \text{ and } 1.5 \le m(T) \le 5.0 \quad \text{(A-7-8)}$$

$f(T)$ = engineering stress at elevated temperatures, ksi (MPa)

$\varepsilon(T)$ = engineering strain at elevated temperatures

$\varepsilon_y(T)$ = strain at the yield stress at elevated temperatures

$$= 0.002 + \frac{F_y(T)}{E(T)} \quad \text{(A-7-9)}$$

$\varepsilon_u(T)$ = ultimate strain at elevated temperatures, which may be approximated as

$$\varepsilon_u(T) = 1 - \frac{F_2(T)}{F_u(T)} \quad \text{(A-7-10)}$$

k_E, k_u, k_y, and k_2 shall be taken from Appendix 4, Table A-4.2.1, Table A-4.2.2, Table A-4.2.3, or Table A-4.2.4 for the relevant type of stainless steel.

COMMENTARY
on the Specification for Structural Stainless Steel Buildings

June 11, 2021

(The Commentary is not a part of ANSI/AISC 370-21, *Specification for Structural Stainless Steel Buildings*, but is included for informational purposes only.)

INTRODUCTION

The Specification is intended to be complete for normal design usage.

The Commentary furnishes background information and references for the benefit of the design professional seeking further understanding of the basis, derivations, and limits of the Specification.

The Specification and Commentary are intended for use by design professionals with demonstrated engineering competence.

COMMENTARY SYMBOLS

The Commentary uses the following symbols in addition to the symbols defined in the Specification. The section number in the righthand column refers to the Commentary section where the symbol is first used.

Symbol	Definition	Commentary Section
A	Cross-sectional area, in.^{2} (mm^2)	C2.3
A_o	Area of rectangular HSS bounded by the midline of the section, in.^{2} (mm^2)	G8
C_{vL}	Shear buckling strength coefficients for round HSS of long length	G4
C_{vM}	Shear buckling strength coefficients for round HSS of intermediate length	G4
C_{v1}	Shear post-buckling strength reduction factor	G2.1
D	Diameter of the cross section, in. (mm)	G4
D	Heat perimeter, in. (m)	App. 4.2.2
F	Fabrication factor	B3.1
F_{cr}	Critical elastic shear buckling stress, ksi (MPa)	G4
$F_{cs,max}$	Maximum compressive stress in the cross section, ksi (MPa)	App. 1.3.3d
F_e	Euler stress, ksi (MPa)	E3
$F_{el,f}^{F}$	Elastic local buckling stress of isolated flange assuming fixed boundary conditions, ksi (MPa)	App. 1.3.3d
$F_{el,f}^{SS}$	Elastic local buckling stress of isolated flange assuming simply supported boundary conditions, ksi (MPa)	App. 1.3.3d
$F_{el,p}^{F}$	Elastic local buckling stress of isolated critical element assuming fixed boundary conditions, ksi (MPa)	App. 1.3.3d
$F_{el,p}^{SS}$	Elastic local buckling stress of isolated critical element assuming simply supported boundary conditions, ksi (MPa)	App. 1.3.3d
$F_{el,w}^{F}$	Elastic local buckling stress of isolated web assuming fixed boundary conditions, ksi (MPa)	App. 1.3.3d
$F_{el,w}^{SS}$	Elastic local buckling stress of isolated web assuming simply supported boundary conditions, ksi (MPa)	App. 1.3.3d
$F_{f,max}$	Maximum compressive stress in the flange, ksi (MPa)	App. 1.3.3d
F_m	Mean value of the fabrication factor	B3.1
$F_{m,des}$	Mean value of the fabrication factor associated with the design thickness	B4.2
$F_{s,el}$	Elastic shear buckling stress, ksi (MPa)	G4
F_{ub}	Tensile strength of the bolt, ksi (MPa)	J3.8
$F_{w,max}$	Maximum compressive stress in the web, ksi (MPa)	App. 1.3.3d
$F_{y,Pin\text{-}A}$	Lesser between the specified minimum yield strength of the pin and connecting part A, ksi (MPa)	J8
$F_{y,Pin\text{-}B}$	Lesser between the specified minimum yield strength of the pin and connecting part B, ksi (MPa)	J8
$F_{0.05\%}$	0.05% offset strength, ksi (MPa)	App. 7.7.1

G	Initial shear modulus, ksi (MPa)	G2.2
G_t	Tangent shear modulus, ksi (MPa)	G2.2
$L_{el,p}$	Elastic local buckling half-wavelength of isolated web and flange, in. (mm)	App. 1.3.3d
$L_{el,f}^{F}$	Elastic local buckling half-wavelength of isolated flange assuming fixed boundary conditions, in. (mm)	App. 1.3.3d
$L_{el,f}^{SS}$	Elastic local buckling half-wavelength of isolated flange assuming simply supported boundary conditions, in. (mm)	App. 1.3.3d
$L_{el,p}^{F}$	Elastic local buckling half-wavelength of isolated critical element assuming fixed boundary conditions, in. (mm)	App. 1.3.3d
$L_{el,p}^{SS}$	Elastic local buckling half-wavelength of isolated critical element assuming simply supported boundary conditions, in. (mm)	App. 1.3.3d
$L_{el,w}^{F}$	Elastic local buckling half-wavelength of isolated web assuming fixed boundary conditions, in. (mm)	App.1.3.3d
$L_{el,w}^{SS}$	Elastic local buckling half-wavelength of isolated web assuming simply supported boundary conditions, in. (mm)	App. 1.3.3d
M	Material factor	B3.1
M_m	Mean value of the material factor	B3.1
M_p	Plastic moment, kip-in. (N-mm)	B4.1
$M_{pin,max}$	Maximum bending moment in a pin, kip-in. (N-mm)	J8
P	Professional factor	B3.1
P_m	Mean value of the professional factor	B3.1
P_y	Squash load, kips (N)	E2
Q_m	Mean value of the load effect	B3.1
R	Radius of round sections, in. (mm)	G4
Ra	Arithmetic mean average surface roughness	A3.1a
R_m	Mean resistance of a structural member	B3.1
R_n	Nominal resistance of the structural elements	B3.1
R_n	Nominal strength of the fillet welded joint specimens, kips (N)	J2.4
Rt	Surface roughness coefficient	J3.8
Rz	Surface roughness coefficient	J3.8
S_B	Stefan-Boltzmann constant = 3.97×10^{-14} Btu/ft-in-s-°F^4 (5.67×10^{-8} W/m^2-°C^4)	App. 4.2.2
S_{xe}	Effective section modulus, in.^{3} (mm^3)	F5
T_F	Temperature of the fire, °F (°C)	App. 4.2.2
T_{FK}	Temperature of the fire, °K	App. 4.2.2
T_S	Temperature of the steel, °F (°C)	App. 4.2.2
T_{SK}	Temperature of the steel, °K	App. 4.2.2
V_F	Geometric coefficient of variation	B3.1
V_M	Material coefficient of variation	B3.1
V_n	Nominal shear strength, kips (N)	G4
V_P	Professional coefficient of variation	B3.1
V_Q	Coefficient of variation of the load effect, Q	B3.1
V_R	Coefficient of variation of the resistance, R	B3.1
W	Weight (mass) per unit length, lb/ft (kg/m)	App. 4.2.2
a	Heat transfer coefficient, Btu/(ft^2-s-°F) (W/m^2-°C)	App. 4.2.2
a_c	Convective heat transfer coefficient	App. 4.2.2

a_r	Radiative heat transfer coefficient	App. 4.2.2
b_e	Effective width, in. (mm)	F5
b_p	Width of element, in. (mm)	App. 1.3.3d
c	Influence coefficient that transfers load intensities to load effects	B3.1
c_s	Specific heat of the steel, BTU/lb-°F (J/kg-°C)	App. 4.2.2
e_0	Equivalent member imperfection amplitude, in. (mm)	App. 1.3.3c
f	Stress, ksi (MPa)	G4
h	Overall height of the section in the direction of bending, in. (mm)	App. 2.6
k	Plate buckling factor	B4.1
k	Buckling coefficient based on the stress distribution, Ψ	App. 1.3.3d
k_{Lb}	Half-wavelength coefficient	App. 1.3.3d
t	Sheet/strip/plate thickness, in. (mm)	A3
t	Thickness of round HSS, in. (mm)	G4
t_{nom}	Nominal thickness, in. (mm)	B4.2
t_p	Thickness of element, in. (mm)	App. 1.3.3d
y_c	Distance from the elastic neutral axis (ENA) to the outer compressive fiber, in. (mm)	App. 2.6
Δt	Time interval, s	App. 4.2.2
α_{eq}	Imperfection factor	App. 1.3.3c
α_{LT}	Lateral-torsional buckling coefficient	F
β	Reliability index	B3.1
β_f	Flange stress correction factor	App. 1.3.3d
β_w	Web stress correction factor	App. 1.3.3d
ε_{csm}	Maximum attainable CSM compressive strain	App. 2.5
$\varepsilon_{csm,t}$	CSM tensile strain	App. 2.6
ε_F	Emissivity of the fire and view coefficient	App. 4.2.2
ζ	Interaction coefficient for HSS and I-sections	App. 1.3.3d
η	Plasticity reduction factor	G2.2
η	Transition function	App. 1.3.3d
λ_p	Limiting width-to-thickness ratio for compact element	B4.1
λ_{pw}	Limiting width-to-thickness ratio for compact web	F4
λ_{rw}	Limiting width-to-thickness ratio for noncompact web	F4
λ_s	Shear buckling slenderness	G4

CHAPTER A
GENERAL PROVISIONS

A1. SCOPE

The basic purpose of the provisions in this Specification is the determination of the nominal and available strengths of the members, connections, and other components of stainless steel building structures.

This Specification is required because the mechanical and physical properties of stainless steels are different from those of the other steel alloys covered by the AISC *Specification for Structural Steel Buildings* (AISC, 2016), hereafter referred to as the 2016 AISC *Specification*. An additional difference is that stainless steel alloys are typically used without protective coatings and are specified based on the level of corrosion resistance that is required for the service environment. The handling, fabrication, cleaning requirements, and physical and mechanical property differences between each stainless steel alloy family and other steel alloys must be considered during design.

This Specification provides two methods of design:

(a) Load and Resistance Factor Design (LRFD): The nominal strength is multiplied by a resistance factor, ϕ, resulting in the design strength, which is then required to equal or exceed the required strength determined by structural analysis for the appropriate LRFD load combinations specified by the applicable building code.

(b) Allowable Strength Design (ASD): The nominal strength is divided by a safety factor, Ω, resulting in the allowable strength, which is then required to equal or exceed the required strength determined by structural analysis for the appropriate ASD load combinations specified by the applicable building code.

This Specification gives provisions for determining the values of the nominal strengths according to the applicable limit states and lists the corresponding values of the resistance factor, ϕ, and the safety factor, Ω. Nominal strength is usually defined in terms of resistance to a load effect, such as axial force, bending moment, shear, or torque, but in some instances it is expressed in terms of a stress. The ASD safety factors are calibrated to give the same structural reliability and the same component size as the LRFD method at a live-to-dead load ratio of 3. The term *available strength* is used throughout the Specification to denote design strength and allowable strength, as applicable.

This Specification is applicable to both buildings and other structures. Many structures found in petrochemical plants, power plants, food and beverage processing, water and wastewater treatment, pharmaceutical plants, pulp and paper plants, and other industrial applications are designed, fabricated, and erected in a manner similar to buildings. It is not intended that this Specification address stainless steel structures with vertical and lateral force-resisting systems that are not similar to buildings, nor those constructed of shells or catenary cables.

The stainless steels considered herein are designated based on one of three alloy families: austenitic, duplex, and precipitation hardening. The austenitic and duplex alloy families are the most commonly used structural stainless steels. Both provide a wide range of corrosion resistance levels, from the common lower alloyed stainless steels to alloys capable of withstanding long-term immersion in seawater or severely corrosive industrial environments.

The most widely used types of austenitic stainless steel are iron based with 16 to 18% chromium, 8 to 14% nickel, and they can have 2 to 3% molybdenum additions. The more corrosion-resistant austenitic stainless steels, which are used in particularly severe service environments, have substantially higher chromium and nickel levels as well as up to 8% molybdenum and nitrogen additions. In comparison to other steel alloys, which have a body-centered cubic atomic (crystal) structure, austenitic stainless steels have a face-centered cubic atomic structure and microstructure of austenite. As a result, austenitic stainless steels, are highly ductile; easily cold formed; nonmagnetic, unless heavily cold worked or cast; and readily weldable. Relative to other steel alloys, they also have significantly better toughness over a wide range of temperatures, including cryogenic. Austenitic stainless steels can be strengthened by cold work but not by heat treatment.

Duplex stainless steels are substantially stronger than annealed austenitic stainless steels and have a mixed microstructure of austenite and ferrite. They are identified as duplex (austenitic-ferritic) stainless steels in the ASTM standards and typically contain 20 to 26% chromium, 1 to 8% nickel, 0.05 to 5% molybdenum, and 0.05 to 0.3% nitrogen. Although duplex stainless steels have good ductility, their higher strength and greater springback make them less formable than austenitic stainless steels. They can be strengthened by cold working but not by heat treatment. They have good weldability and better resistance to chloride stress corrosion cracking and crevice corrosion than comparable austenitic stainless steels.

Prequalified welding procedure specifications (PWPS) for welding the most commonly used austenitic stainless steels and for welding austenitic stainless steel to other steels can be found in AWS D1.6/D1.6M (AWS, 2017). Although they are regularly welded, AWS does not have PWPS for the highly alloyed austenitic stainless steels or the duplex stainless steels; alloy-specific advice should be obtained, for example, from the stainless steel mill or stainless steel welding product suppliers.

Precipitation hardening stainless steels are the least corrosion resistant and least ductile alloy family in this Specification and are only considered appropriate for rural locations without pollutants, farm chemical or salt exposure, and light industrial exposure. They are not suitable for coastal, deicing salt, or swimming pool building applications. They are generally not welded but provide higher strength than the austenitic and duplex stainless steel alloys. High-strength fasteners and tension bars are sometimes made from precipitation hardening stainless steels. This Specification only includes two alloys, S15500 and S17400 (17-4 PH), which are martensitic precipitation hardening stainless steels. Precipitation hardening heat treatment is necessary for these alloys, but the allowable options included in this Specification have been deliberately limited to those with the greatest ductility and the least susceptibility to hydrogen embrittlement and sulfide stress cracking. While higher strengths can be obtained with other heat treatments, they are strongly discouraged due to their known risk of hydrogen embrittlement and sulfide stress cracking.

While both non-precipitation hardening martensitic stainless steels and stainless steels with dual-phase microstructures that are mainly ferritic with some martensite [i.e., ASTM A1010/A1010M (ASTM, 2018e) or ASTM A709/A709 Grade 50CR (345CR) (ASTM, 2018f)] have been used for structural applications, there was not sufficient published structural research at the time this Specification was developed to justify their inclusion.

The design of steel plate clad with stainless steel is not covered by this Specification. This product does not behave like stainless steel.

Headed stud anchors are not covered by this Specification because industry-specific design specifications do not currently exist.

1. Seismic Applications

Seismic response modification coefficients have not been developed for structural stainless steel systems; with proper care in alloy selection and connection detailing, structural stainless steel systems are capable of meeting or exceeding the ductility of other structural steel seismic systems.

Studies on the behavior of stainless steel subject to cyclic loading have shown that the increased ductility of stainless steel enables greater dissipation in energy during the course of a seismic event (di Sarno et al., 2003; Ye et al., 2006; Nip et al., 2010; Wang et al., 2014a; Chacón et al., 2018).

A2. REFERENCED SPECIFICATIONS, CODES, AND STANDARDS

Section A2 provides references to documents cited in this Specification. The date of the referenced document found in this Specification is the intended date referenced in this Commentary unless specifically indicated otherwise. Note that not all alloys in a particular material specification are necessarily approved for use according to this Specification. For a list of approved materials and grades, see Section A3.

A3. MATERIAL AND PRODUCT ORDER REQUIREMENTS

1. Structural Stainless Steel Materials

The unified numbering system (UNS) for metals and alloys is used for identifying all stainless steels in ASTM standards per agreement between SAE International and ASTM International. The numbers are assigned by SAE International (SAE, 2017), and the system is governed by two standards: SAE J1086 (SAE, 2012) and ASTM E527 (ASTM, 2016a). The UNS designation is not a specification because it does not establish requirements for form, condition, properties, or quality. It is a unified identifier for establishing the overall composition maximums or ranges for unique metal and alloy chemistries, but the controlling limits of chemistries for specific product forms and applications are established in ASTM standards and other specifications. Each UNS designation consists of a single-letter prefix followed by five digits. “S” is the prefix for stainless steels and “N” for nickel alloys. These alloy composition requirements may have variations among standards when the ASTM A01 Committee on Steel has found it necessary for specific products.

When a common or proprietary name exists, it may be indicated in addition to the specified UNS designation. Many alloys do not have common names, and there may be more than one UNS designation associated with a common name; for example, UNS S32205 and S31803 are both sometimes called 2205 even though the alloy compositions and corrosion resistance are different.

1a. Service Environment Assessment

Stainless steel is almost always specified for corrosion resistance either to retain functional or structural integrity over the expected service life, or to meet aesthetic requirements with little or no maintenance. In many cases, it is best for a metallurgical engineer with specific expertise in stainless steel to assist in selecting an appropriate alloy(s).

The corrosion performance of stainless steels in a given environment is determined by several factors. The alloy composition, specifically the levels of chromium, molybdenum, and nitrogen, can indicate the relative corrosion resistance of the alloy. This should not be the only consideration. The stainless steel alloy family must be considered when chloride stress corrosion cracking, crevice corrosion, and other service environment or design considerations can affect performance. Additionally, other factors such as surface finish, unsealed crevices, and surface deposits should be considered.

It may be necessary to use different stainless steels in different areas on the same structure. This is not generally a problem as long as each stainless steel used has adequate corrosion resistance for the environment to which that part is exposed. Using more corrosion resistant materials for fittings, fasteners, and other small parts is a common practice.

Table C-A3.1 lists common service environment characteristics. It is best practice to assess and, when possible, quantify all service environment characteristics that can affect the corrosiveness of the service environment in order to accurately determine the risk of different types of corrosion (i.e., atmospheric, industrial, aqueous, crevice, galvanic, microbiological, etc.). Without this information, it is difficult to accurately determine the most appropriate stainless steel alloy family or the minimum stainless steel corrosion resistance necessary to meet project performance requirements. The variability of service environment characteristics should also be considered. Variations in chemistry, operating temperature, humidity, pH, and other factors can dramatically change suitable alloy choices. These characteristics are important for selecting any material, or, in the case of other steel alloys, appropriate coating systems for withstanding a corrosive service environment. The guidance on corrosion in AISC Design Guide 27, *Structural Stainless Steel* (Baddoo and Meza, 2021), hereafter referred to as AISC Design Guide 27, assumes that the engineer has assessed and quantified the service environment.

TABLE C-A3.1
Typical Data Used for Corrosion Assessment in Common Exposure Environments

Atmospheric/Non-Immersed Environments
Urban pollution exposure level (1)
Industrial pollution exposure level and source (1)
Particulate level and type (industrial, dust, etc.) (1)
Rain acidity (2)
Average multi-year total wet and dry chloride salt deposition level (2)
Deicing exposure level and type of chloride salts used[a] (3)
Average monthly daytime and evening temperatures (4)
Average monthly daytime and evening relative humidity (4)
Frequency and quantity of total precipitation, heavy rain, fog, and snow (4)
Expected manual cleaning frequency
Whether there is exposure to precipitation[a]
Continually or Regularly Immersed Environments
Chemical composition range of liquid and atmosphere, if industrial
If not continuously immersed, frequency of immersion
pH range of liquid
Chlorine and/or bromine and chloride concentration, if used
Temperature range of liquid and atmosphere
Presence of microbiological organisms for natural water
Salinity of liquid and atmospheric environment
Industrial Non-Immersed Environments
Gas composition or atmospheric exposure
Operating temperature range
Presence and concentration of sulfides
Chemicals and their composition range
Salinity
Humidity/moisture exposure
Whether there is exposure to precipitation[b]

[a] The elevation and distance from the structure to streets and highways is an important consideration.
[b] Rainfall can wash exposed stainless steel and reduce the severity of corrosion. This is especially true in coastal exposures where salt fogs can cause corrosion. Areas exposed to the air but not exposed to rain, such as train stations, bus shelters, industrial equipment shelters, soffits, and the undersides of marquees are especially subject to corrosion.

Data sources:
(1) U.S. and state Environmental Protection Agency data or other government sources.
(2) U.S. National Atmospheric Deposition Program data provides rain acidity, coastal salt deposition, and some pollutant data, as do other national assessment programs.
(3) Municipal or state reports.
(4) Weatherbase.com provides detailed global weather data.

There is a direct correlation between corrosion performance and finish. The use of a rougher finish, or one with mill scale or heat tint, in a corrosive environment can make the specification of a more corrosion resistant stainless steel or more regular cleaning necessary. In clean room, pharmaceutical, food processing, and other applications where surface sanitation is critical, finish texture determines cleanability.

Finish texture is also important for aesthetic applications. The expected corrosion performance of the alloy/finish texture combination determines if aesthetic objectives will be achieved without extensive cleaning.

The arithmetic mean average surface roughness, *Ra*, is often used when specifying for improved corrosion resistance because *Ra* and gloss (i.e., specular reflectivity) are the easiest aspects of surface texture to measure during finish processing whether at a mill or toll polisher (i.e., firm that only provides finishing services). Other finish texture measurements can sometimes be useful for predicting corrosion performance and appearance consistency. Finish is discussed in more detail in AISC Design Guide 27.

1b. ASTM Standards and Minimum Order Requirements

There are hundreds of stainless steel alloys. This Specification lists those products and alloys that are commonly useful to structural engineers, have been researched, and have a history of satisfactory performance. Other stainless steel alloys may be suitable for specific applications, but the evaluation of those stainless steel alloys is the responsibility of the engineer specifying them. The availability of products made from specific alloys or in some sizes may be limited and should be confirmed prior to specification. Some manufacturers regularly make short runs of specialized products but it may be more expedient and cost effective to use off-the-shelf higher alloyed or larger products.

Table User Note A3.1 lists some physical properties that can be used for design. The values for modulus of elasticity and density are taken from ASME (2019) and the values for thermal expansion are taken from CEN (2014). Test data for austenitic stainless steel that has undergone severe strain hardening, have shown a reduced modulus of elasticity (Manninen et al., 2011). The relationship between the degree of cold work and tensile properties of austenitic stainless steels is not a simple one and depends on the chemical composition of the alloy. Additionally, the amount of work hardening is not uniform throughout the product: the material near the surfaces reaches higher strength as it work-hardens more than the material in the center.

Tables C-A3.2 to C-A3.6 give specified specified mechanical properties for stainless steel products covered in Table A3.1 according to the relevant ASTM standards.

TABLE C-A3.2
Minimum Mechanical Property Requirements for Plate, Sheet, and Strip ASTM A240/A240M (ASTM, 2020f)

UNS Designation	Type	Tensile Strength, F_u		Yield Strength, F_y		Elongation in 2 in. (50 mm)
		ksi	MPa	ksi	MPa	%
Austenitic						
N08367:	–					
Sheet/strip		100	690	45	310	30
Plate		95	655	45	310	30
N08904	904L	71	490	31	220	35
N08926	–	94	650	43	295	35
S30400	304	75	515	30	205	40
S30403	304L	70	485	25	170	40
S31600	316	75	515	30	205	40
S31603	316L	70	485	25	170	40
S31703	317L	75	515	30	205	40
S32100	321	75	515	30	205	40
S31254:	–					
Sheet/strip		100	690	45	310	35
Plate		95	655	45	310	35
Duplex						
S32003:	–					
$t > 0.187$ in. (5 mm)		95	655	65	450	25
S32101	–	94	650	65	450	30
S32202	–	94	650	65	450	30
S32205	2205	95	655	65	450	30
S32304	2304	87	600	58	400	25
S32750	2507	116	795	80	550	15
S32760	–	108	750	80	550	25
S82011:	–					
$t > 0.187$ in. (5 mm)		95	655	65	450	30
S82441:	–					
$t < 0.4$ in. (10 mm)		107	740	78	540	25
$t \geq 0.4$ in. (10 mm)		99	680	70	480	25

Only thicknesses above 0.187 in. (5 mm) are shown.

TABLE C-A3.3
Minimum Mechanical Property Requirements for Hollow Structural Sections ASTM A554 (ASTM, 2016b)

UNS Designation	Type	Tensile Strength, F_u		Yield Strength, F_y		Elongation in 2 in. (50 mm)
		ksi	MPa	ksi	MPa	%
Austenitic						
S30403 & S31603	MT 304L MT 316L	70	485	25	170	35
All other austenitic stainless steels	–	75	515	30	205	35
Duplex						
S32003: $t \le 0.187$ in. (5 mm) $t > 0.187$ in. (5 mm)	–	 100 95	 690 655	 70 65	 485 450	 25 25
S32101: $t < 0.4$ in. (10 mm) $t \ge 0.4$ in. (10 mm)	–	 101 94	 700 650	 77 65	 530 450	 30 30
S32202	–	94	650	65	450	30
S32205	2205	95	655	65	450	25
S32304	2304	87	600	58	400	25
S32750	2507	116	795	80	550	15
S32760	–	108	750	80	550	25
S82011: $t \le 0.187$ in. (5 mm) $t > 0.187$ in. (5 mm)	–	 101 95	 700 655	 75 65	 515 450	 30 30
S82441: $t < 0.4$ in. (10 mm) $t \ge 0.4$ in. (10 mm)	–	 107 99	 704 680	 78 70	 540 480	 25 25

TABLE C-A3.4
Minimum Mechanical Property Requirements for Bars and Shapes ASTM A276/A276M (ASTM, 2017a)

Designation	Condition (A, S, or B), finish, and size [in. (mm)]	Tensile Strength, F_u		Yield Strength, F_y		Elongation in 2 in. (50 mm)
		ksi	MPa	ksi	MPa	%
Austenitic						
N08367	A, HF or CF	95	655	45	310	30
N08904 904L	A, HF or CF	71	490	31	220	35
N08926	A, HF or CF	94	650	43	295	35
304, 316	A, HF	75	515	30	205	40
304, 316	A, CF:					
	≤ ½ (13)	90	620	45	310	30
	> ½ (13)	75	515	30	205	40
304L, 316L	A, HF	70	485	25	170	40
304L, 316L	A, CF:					
	≤ ½ (13)	90	620	45	310	30
	> ½ (13)	70	485	25	170	40
304, 304L, 316, 316L	S, CF:					
	> 1½ (38) ≤ 1¾ (44)	95	655	45	310	28
	> 1¾ (44) ≤ 2 (50)	95	650	75	515	25
	> 2 (50) ≤ 2½ (63)	90	620	65	450	30
	> 2½ (63) ≤ 3 (75)	80	550	55	380	30
304, 304L, 316, 316L	B, CF:					
	≤ ¾ (19)	125	860	100	690	12
	> ¾ (19) ≤ 1 (25)	115	795	80	550	15
	> 1 (25) ≤ 1¼ (31)	105	725	65	450	20
	> 1¼ (31) ≤ 1½ (38)	100	690	50	345	24

TABLE C-A3.4 (continued) Minimum Mechanical Property Requirements for Bars and Shapes ASTM A276/A276M (ASTM, 2017a)

Designation	Condition (A, S, or B), finish, and size [in. (mm)]	Tensile Strength, F_u		Yield Strength, F_y		Elongation in 2 in. (50 mm)
		ksi	MPa	ksi	MPa	%
Duplex						
S32101	A	94	650	65	450	30
S32202	A	94	650	65	450	30
S32205	A	95	655	65	450	25
S32304	A	87	600	58	400	25
S32750	A: ≤ 2 (50)	116	800	80	550	15
	A: > 2 (50)	110	760	75	515	15
S32760	A	109	750	80	550	25
S32760	S	125	860	105	720	16
S82441	A: < 7⁄16 (11)	107	740	78	540	25
	A: ≥ 7⁄16 (11)	99	680	70	480	25

A = annealed, S = strain hardened (relatively light cold work), B = relatively severe cold work, HF = hot finished, CF = cold finished

Table C-A3.5 Minimum Mechanical Property Requirements for Precipitation Hardening Stainless Steels Bars and Shapes ASTM A564/A564M (ASTM, 2019g)

UNS (Common Name)	Heat Treatment Condition	Tensile Strength, F_u		Yield Strength, F_y		Elongation in 2 in. (50 mm) or 4D[a]
		ksi	MPa	ksi	MPa	%
S15500[b] (XM-12)	H1150	135	930	105	725	16
	H1150M	115	795	75	520	18
S17400[b] (630)	H1150	135	930	105	725	16
	H1150M	115	795	75	520	18
	H1150D	125	860	105	725	16

[a] D = diameter of the bar
[b] These precipitation hardening stainless steels in conditions other than H1150 present a significant risk of hydrogen embrittlement and are not recommended for structurally critical applications.

TABLE C-A3.6
Minimum Mechanical Property Requirements for Laser and Laser Hybrid Welded Bars, Plates, and Shapes ASTM A1069/A1069M (AISC, 2019f)

Strength Grade	Tensile Strength, F_u		Yield Strength, F_y		Elongation in 2 in. (50 mm)
	ksi	MPa	ksi	MPa	%
1	Refer to the minimum mechanical property requirements in ASTM A240/A240M, A276/A276M, A554, or A479/A479M (ASTM, 2020g)				
Austenitic					
2[a]	80	550	35	240	35
Duplex					
3[b]	95	655	65	450	25
4[c]	116	800	80	550	15

[a] Applies to S30403 (304L), S31603 (316L), and S31703 (317L). Other austenitics should be ordered to strength Grade 1, ASTM A240/A240M, mechanical properties.
[b] Applies to S32205 (2205) up to 2½ in. (63 mm) in thickness.
[c] Applies to the more highly alloyed duplexes, S32750 and S32760, up to 2 in. (50 mm) in thickness.

The stainless steel specifications do not always list the strength levels for different thicknesses or diameters so the minimum requirements may be indicative of the properties for the heaviest thickness or diameter products, which have the lowest yield strength. For that reason, stainless steels often have a much larger margin between the actual and minimum yield strengths than would be expected with other steel alloys, particularly for plate. This is particularly true for ASTM A240/A240M (ASTM, 2020f) plate, which is used for a wide range of applications, and some can have lower yield strength requirements. The mill certificates list the actual mechanical properties. For plate thicknesses up to 1 in. (25 mm), the measured yield strength of austenitic stainless steels may be 25 to 40% above the minimum and up to 20% above the minimum for duplex stainless steels. For thicknesses of 2 to 2.5 in. (50 to 63 mm) and above, the listed values are usually only slightly above the ASTM specified minimum yield strength. ASTM A1069/A1069M (ASTM, 2019f) includes higher strength grade level options for common alloys for thicknesses under 2 in. (50 mm).

The "L" in some austenitic stainless steel designations indicates a low carbon version. The "L" grades carry a yield strength penalty of 5 ksi (35 MPa). This strength penalty can be avoided by specifying "dual-certified" material (e.g., S30400/S30403 or S31600/S31603). Certification to both designations provides material with the higher strength of S30400 or S31600 and the lower carbon content of S30403 or S31603.

Table A3.1 lists the minimum order requirements for a stainless steel product given in the relevant ASTM standard. Stainless steel is a specialized material, and ASTM standards have more minimum order requirements for stainless steel than other steel alloys because of the inherent differences between the steels and how they are used. Most stainless steel standards have lists of optional requirements that can be stipulated to meet specific service or application requirements.

Specification of the stainless steel alloy family and the alloy chemistry are critical because they determine corrosion resistance, strength, machinability, formability, fire performance, corrosion resistance to specific environmental conditions, and other critical characteristics.

"Condition" indicates the required heat treatment or level of cold work and the strength level.

Finish specification is critical for corrosion performance and for applications with surface sanitation requirements.

At the time of issuance of this Specification, hollow structural sections (HSS), pipe, bar, hot-rolled, and laser or laser hybrid welded shapes are standard stocked items. The range of stocked sizes and shapes is more limited than for other structural steels and can be limited to a few alloys. It is important to determine the availability of shapes in the desired alloy, and, if necessary, the production lead times early in the design process. Acceptable alloy substitutions should be stipulated. If an acceptable alloy and the product forms necessary to make the shape are stocked, lead times are usually a few weeks.

The most commonly stocked hot-rolled profiles are up to 6 in. (150 mm) in size, but larger sizes are inventoried, such as C8×18.75 (C203×476) channel. Extruded shapes are readily available, and typically, the shape must fall within an 8-in. (200-mm) diameter, but that will vary with the alloy and supplier. Laser and laser hybrid sections are commonly stocked, including sizes of up to 8-in. (200-mm) angle, beams up to 50 in. (1 250 mm) tall, and channels up to C15×33.9 (C380×861); mill orders of other standard and custom shapes typically require a few weeks lead time. Structural shapes welded by other processes are also readily obtainable and stocked by some service centers, including large and heavy sections, but there is no ASTM product standard; therefore, certification, reporting, and inspection requirements should be stipulated in accordance with this Specification.

(a) Hot-Rolled and Extruded Structural Shapes

Extruding is typically used to create custom shapes to minimize machining. The maximum size is determined by the alloy. Currently, the largest extrusions must fit within an 8-in. (200-mm) diameter. Shapes are descaled by machining, grinding, blasting, or pickling. Regarding finish requirements, either Class A or Class C surface preparation should be stipulated on the purchase order. Class A consists of grinding for the removal of imperfections of a hazardous nature, such as fins, tears, and jagged edges, provided the underweight tolerance is not exceeded and the maximum depth of grinding at any one point does not exceed

10% of the thickness of the section. Class C consists of grinding for the removal of all visible surface imperfections, provided that the underweight tolerance is not exceeded and the maximum depth of grinding at any point does not exceed 10% of the thickness of the section.

The hot-rolled and extruded structural shape dimensional tolerances in ASTM A484/A484M (ASTM, 2020c) are currently limited to hot-finished equal and unequal leg angles (all sizes) and channels and tees with a depth and width less than 3 in. (75 mm). All other hot-rolled shape tolerances are in ASTM A484/A484M, Table 16. The ASTM A484/A484M tolerances are generally looser than those in ASTM A6/A6M (ASTM, 2019c) and do not cover as broad a size and shape range. The engineer can specify compliance with the shape tolerances in ASTM A6/A6M, Table 16, Table 17, or Table 18, but acceptance is subject to contractual agreement between the purchaser and supplier. No part of ASTM A6/A6M, other than dimensional tolerances, is applicable because it is intended for other structural steels.

(b) Hollow Structural Sections (HSS)

ASTM A554 (ASTM, 2016b) is the welded mechanical tubing specification for HSS. It is the preferred HSS specification because it does not require pressure and other testing. While it includes HSS up to 16 in. (400 mm), larger sections made to the requirements of this specification can be ordered. When duplexes are ordered to ASTM A554, a filler metal should be required in the project specification if the weld is expected to have corrosion resistance that is equivalent to the base metal. ASTM A554 does not currently provide that option. Laser and laser hybrid welded ASTM A1069/A1069M (ASTM, 2019f) HSS is an alternative to ASTM A554. Products manufactured in accordance with ASTM A554 are available in round, square, rectangular, and custom shapes. ASTM A1069/A1069M includes square and rectangular HSS, as well as custom shapes, and has four sharp welded square corners while ASTM A554 product corners are rounded. ASTM A554 provides the dimensional tolerance requirements for either product.

Round HSS certified to ASTM A269/A269M (austenitic stainless steel) (ASTM, 2019a) and ASTM A789/A789M (duplex stainless steel) (ASTM, 2018a) may be more readily available in some sizes and are acceptable substitutes. These standards have more stringent requirements than ASTM A554 because of their use for carrying pressurized fluids.

For rounds, the outside diameter and nominal thickness of the wall should be specified. For the cold-reduced condition, outside and inside diameter, or inside diameter and wall dimensions, should be specified.

ASTM A554 products can be supplied in the as welded, welded and annealed, cold reduced, or cold-reduced and annealed condition. The standard finish for ASTM A554 products is free of scale. If other conditions are required they should be specified.

The default finish for ASTM A1069/A1069M products is descaled and passivated in accordance with ASTM A380/A380M (ASTM, 2017c), but an as-welded finish, or a finish pickled in accordance with ASTM A380/A380M can also be specified.

Finish requirements should be carefully considered. In the as-welded condition, the welds have reduced corrosion resistance because the heat tint has not been removed and corrosion resistance restored. For improved weld corrosion resistance, the pickled or descaled and passivated finish conditions should be considered. Even if there will be subsequent polishing, these finishes will provide better weld corrosion resistance.

Regarding pipe, ASTM A312/A312M (ASTM, 2019b) addresses austenitic stainless steel and ASTM A790/A790M (ASTM, 2020b) addresses duplex stainless steel pressure-rated round pipe. The added testing requirements for pressure-rated pipe make this product more expensive than HSS produced to ASTM A554, but it may be more readily available in larger sizes and heavier thicknesses. Dimensional tolerances are determined by ASTM A999/A999M (ASTM, 2018g). The specification requires that the outside diameter and nominal thickness of the wall be specified.

Special finishes for HSS and pipe, such as polishing, can be obtained but are typically applied by a polishing company after material production.

(c) Built-Up Shapes—Laser and Laser Hybrid

ASTM A1069/A1069M is a structural section product standard covering built-up sections such as HSS, I-shaped members, channels, angles, and custom shapes. The chemical composition requirements are determined by the raw material standard, which could be ASTM A240/A240M, ASTM A276/A276M, or ASTM A479/A479M. Most sections produced to this standard are made with ASTM A240/A240M plate. AWS D1.6/D1.6M does not address laser or laser hybrid welds. ASTM A1069/1069M was developed to fill that gap and provide a high quality product.

ASTM A1069/A1069M requires the laser and laser hybrid welding procedure, operator, and production line to be prequalified in accordance with either the relevant International Standards Organization (ISO) or AWS standards. This prequalification requires documentation that the laser or laser hybrid welds provide strength and corrosion resistance that are equal to or greater than the base material. The purchaser can request additional optional testing to document this on their order.

ASTM A1069/A1069M offers four strength grades. Grade 1 provides the minimum strength levels in ASTM A240/A240M for any austenitic or duplex stainless steel alloy produced to this standard. Purchasers can stipulate Grade 2 for the following austenitic stainless steels: S30403 (304L), S30409 (304H), S31603 (316L), S31653 (316LN), and S31703 (317L). They may obtain a higher strength level than the minimum requirements of ASTM A240/A240M;

specifically a specified minimum yield strength of 35 ksi (240 MPa) and specified minimum tensile strength of 80 ksi (550 MPa). Grades 3 and 4 allow S32205 to be ordered to a higher strength level than ASTM A240/A240M up to a thickness of 2.5 in. (63 mm), and either S32750 or S32760 can be ordered to a higher strength up to 2 in. (50 mm).

The specification for a product to ASTM A1069/A1069M should include the following items:

(1) Name of structural product

(2) Shape designation and applicable dimensions including size, thickness, width, diameter, and length, if applicable

(3) UNS designation

(4) Quantity (weight or number of pieces)

(5) Condition of welded product, whether as welded, or subsequently stress-relieved, or heat treated

(6) Finish options:
- As welded
- Pickled in accordance with ASTM A380/A380M
- Descaled and passivated in accordance with ASTM A380/A380M

When corrosion testing of duplex stainless steel welds is required, Standard Test Method ASTM A1084 (ASTM, 2015) is used for the lean duplex steels, S32101, S32202, S82011, and S32304, and Standard Test Method ASTM A923 (ASTM, 2014b) is used for the higher alloyed duplex steels S32003, S32205, S32760, S32750, and S82441.

(d) Built-Up Shapes—Other Than Laser and Laser Hybrid

Laser and laser hybrid welded structural sections are not the only welded structural sections produced in dedicated manufacturing facilities. These purchased shapes could be from a dedicated manufacturer of structural sections or purchased from a fabricator who subcontracted the work. Dedicated producers of structural stainless steel shapes have been selling into the U.S. market with test certifications to ISO/EN standards, but those standards are not recognized by this Specification and therefore do not necessarily meet the requirements in Table A3.1(d). Dedicated manufacturers of welded structural shapes can certify to the list of requirements in Table A3.1(d) rather than an international standard. A service center or broker should provide the mill certifications documenting that the requirements of Table A3.1(d) have been met.

Fabricators should have relevant experience with the stainless steel alloy and welding method. Qualification of the welder should be in accordance with AWS B2.1/B2.1M (AWS, 2014). As required in Section N3.2, current certifications for the welders and approved welding procedures should be provided. When the service environment is corrosive, weld corrosion testing of duplex stainless steels to qualify the welder should be considered in accordance with Section J2.

Where ASTM product specifications, such as ASTM A6/A6M (ASTM, 2019c) or AWS D1.6/D1.6M do not provide requirements for member dimensional tolerances, AWS D1.1/D1.1M, clause 7.22, should be used (AWS, 2020).

In the absence of an ASTM product standard, the minimal requirements for product specification and certification by a dedicated manufacturer have been provided.

For corrosion testing of duplex stainless steel welds, see Commentary to Table A3.1(c).

(e) Plate, Sheet, Strip

ASTM A480/A480M (ASTM, 2020a) includes all of the general requirements for sheet, strip, and plate, including, but not limited to, condition, finish, dimensional tolerances, shipping, handling, and other requirements.

The minimum order requirements are:

(1) Quantity (weight and number of pieces)

(2) Name of material (stainless steel)

(3) Condition (hot rolled, cold rolled, annealed, heat treated)

(4) Finish (described below)

(5) Form (plate, sheet, or strip)

(6) Dimensions (decimal or fractional thickness, width, and length)

(7) Edge, strip only (see ASTM A480/A480M, Section 14, for cold-rolled strip)

(8) UNS designation

(9) Specification and date of issue

Special and optional requirements options include:

(1) Improved flatness

(2) Restrictions (if desired) on methods for determining yield strength

(3) Marking requirements

(4) Preparation for delivery

(5) Magnetic permeability

Unless otherwise specified, the standard flatness requirements of ASTM A480/A480M are assumed. The ASTM standard provides tables that include both the standard and stretcher-leveled standard of flatness for wide cold-rolled coil-processed product cut to length (Table A2.8) and hot-rolled wide coil-processed product in cut lengths (Table A2.16). If the sheet/strip/plate is not shipped by the mill to a tighter flatness requirement, the service center can have stretcher leveling done on smaller orders.

While only standard tolerances are listed for mill plate, special processing to obtain tighter flatness tolerances is usually possible but must be agreed with the mill while ordering. The permitted variations in width and length are not the same for standard and stretcher-leveled standard of flatness and these tolerance differences should be considered.

While coatings and fillers can be helpful in achieving visual flatness in other steel alloys, they are not an option for bare stainless steel. If a higher level of flatness or other tighter tolerances are required for fit up or aesthetic appearance, the mill product should be ordered to those tolerances.

ASTM A480/A480M has non-legally binding notes providing general finish descriptions, which are provided in the following discussion. The typical surface roughness is provided for guidance and is not mandatory. The project specification should explicitly include any surface roughness requirements. ASTM A480/A480M does not include all possible finishes. The availability of a special finish will vary with the product. Grinding or polishing of plate is a special finish requirement. Some of the sheet finishes in ASTM A480/A480M are typically applied by a toll polisher instead of the mill. The relationship between surface texture and corrosion resistance is discussed in AISC Design Guide 27.

Descriptions of ASTM A480/A480M Sheet Finishes:

No. 1—Commonly referred to as hot-rolled annealed and pickled or descaled. This is a dull, nonreflective finish.

No. 2D—A smooth, nonreflective cold-rolled annealed and pickled or descaled finish. This nondirectional finish is favorable for retention of lubricants in deep drawing applications.

No. 2B—A smooth, moderately reflective cold-rolled annealed and pickled or descaled finish typically produced by imparting a final light cold-rolled pass using polished rolls. This general-purpose finish is more readily polished than No. 1 or 2D finishes. Product with 2B finish is normally supplied in the annealed plus lightly cold-rolled condition unless a tensile-rolled product is specified.

Bright Annealed Finish—A smooth, bright, reflective finish typically produced by cold rolling followed by annealing in a protective atmosphere so as to prevent oxidation and scaling during annealing.

No. 3—A linearly textured finish that may be produced by either mechanical polishing or rolling. Average surface roughness (*Ra*) may generally be up to 40 μin. (1 000 μmm). A skilled operator can generally blend this finish. Surface roughness measurements differ with different instruments, laboratories, and operators. There may also be overlap in measurements of surface roughness for both No. 3 and No. 4 finishes.

No. 4—A linearly textured finish that may be produced by either mechanical polishing or rolling. Average surface roughness (*Ra*) may generally be up to 25 μin. (625 μmm). A skilled operator can generally blend this finish. Surface roughness measurements differ with different instruments, laboratories, and operators. There may also be overlap in measurements of surface roughness for both No. 3 and No. 4 finishes.

No. 6—This finish has a soft, satin appearance typically produced by tampico (i.e., brushes made from the Tampico plant) brushing a No. 4 finish.

No. 7—Has a high degree of reflectivity. It is produced by buffing a finely ground surface, but the grit lines are not removed. It is chiefly used for architectural or ornamental purposes.

No. 8—This is a highly reflective, smooth finish typically produced by polishing with successively finer grit abrasives, then buffing. Typically, very faint buff of polish lines may still be visible on the final product. Blending after part assembly may be done with buffing.

TR Finish—The finish resulting from the cold rolling of an annealed and descaled or bright annealed product to obtain mechanical properties higher than that of the annealed condition. Appearance will vary depending upon the starting finish, amount of cold work, and the alloy.

Architectural Finishes—Sometimes described as a No. 5 finish, these are a separate category and may be negotiated between buyer and seller, as there are many techniques and finish variations available throughout the world.

Descriptions of ASTM A480/A480M Strip Finishes:

No. 1—Appearance of this finish varies from dull gray matte finish to a fairly reflective surface, depending largely upon composition. This finish is used for severely drawn or formed parts, as well as for applications where the brighter No. 2 finish is not required, such as parts for heat resistance.

No. 2—This finish has a smoother and more reflective surface, the appearance of which varies with composition. This is a general-purpose finish, widely used for household and automotive trim, tableware, utensils, trays, and so forth.

Bright Annealed Finish—See sheet finishes.

TR Finish—See sheet finishes.

Descriptions of ASTM A480/A480M Plate Finishes:

Hot Rolled or Cold Rolled, and Annealed or Heat Treated—The mill scale is not removed from this finish, making it the lowest cost mill finish. This finish is generally only used for heat-resisting applications. The presence of mill scale significantly reduces corrosion resistance and is not acceptable for most stainless steel applications.

Hot Rolled or Cold Rolled, and Annealed or Heat Treated, and Blast Cleaned or Pickled—This is the most common mill finish. The mill scale has been removed. It is used for common corrosion-resisting and most heat-resisting applications and is essentially the same as a No. 1 Finish. This is a rough, uneven finish that typically has grinding marks and other finish inconsistencies. Specifications allow defect removal by grinding as long as the local thickness does not go below the specified minimum. Additional treatments, such as polishing or grinding the entire plate surface can usually visually blend in these areas. Abrasive blasting without first grinding or polishing the entire plate typically does not adequately blend in mill grinding marks for an aesthetic application. If it is important to avoid grinding marks, that information should be in the purchase order.

Hot Rolled or Cold Rolled, and Annealed or Heat Treated, and Surface Cleaned and Polished—This polished finish is much more consistent in appearance and smoother providing improved corrosion resistance. It can be equivalent to a rough No. 3 finish to a No. 4 finish, depending on project specification requirements.

Hot Rolled or Cold Rolled, and Annealed or Heat Treated, and Descaled, and Temper Passed—This is a smoother finish for specialized applications.

Hot Rolled or Cold Rolled, and Annealed or Heat Treated, and Descaled; and Cold Rolled, and Annealed or Heat Treated, and Descaled, and Optionally Temper Passed—This is a smoother finish with fewer surface imperfections.

(f) Hot- and Cold-Finished Bar and Flat Bar Cut from Strip or Plate

The bar used for most structural applications is typically dual certified to both ASTM A276/A276M (ASTM, 2017a) and ASTM A479/A479M (ASTM, 2020g). The specifications are interchangeable for most structural applications, but ASTM A276/A276M is typically the most appropriate specification. ASTM A479/A479M has supplemental requirements (S1, S2, S4, and S5) that could be specified if the bar will be used under one of the following special conditions: high-temperature service (S1), intergranular corrosion testing is required (S2), high cycle fatigue service (S4), and optimal resistance to chloride stress corrosion cracking (S5).

Specification of one of the following conditions is necessary and each is associated with mechanical requirements in ASTM A276/A276M:

Condition A—annealed

Condition S—strain hardened with relatively light cold work

Condition B—relatively severe cold work

ASTM A484/A484M has many finish options, and the options vary with the bar type. The standard finish options are given in the following, but other finishes are also available:

Flat bars cut from strip or plate: Two surfaces pickled or descaled, and two cut surfaces; descaled or pickled (if heat treated after cutting)

Hot-finished bar: As hot finished; as annealed; descaled; rough turned; machine straightened; centerless grinding; polished

Cold-finished bar: Light cold drawing; burnishing; centerless grinding; polishing

Hot-finished bar is usually cleaned by blasting or pickling, or some other descaling method, but will be quite rough. Ground or polished bar will have a smoother surface and better corrosion performance. Grinding is used to improve dimensional tolerances and is usually preferred for tension bars. The finish must be specified.

2. Castings and Forgings

Design and fabrication of cast and forged steel components are not covered in this Specification.

A different naming system is used for castings. The properties of castings may be different from their rolled versions, for example, austenitic stainless steel castings may be slightly magnetic. Further guidance is available from The Steel Founders' Society of America (SFSA) (SFSA, 1995) and Nickel Institute (Houska, 2012).

The choice of casting or forging finish is at the discretion of the manufacturer unless specified in the purchase order. They are typically machined, blasted, or pickled, so mill scale is removed. Machining is typically only done to achieve tolerances; it provides the smoothest finish option.

ASTM standards do not cover casting or forging design, nor do they address manufacturing processes. Mechanical properties, weldability, nondestructive examination, and other requirements should be considered to achieve the level of service expected for the application.

3. Bolts, Washers, and Nuts

The Commentary to Section J3.1 gives some further information on the scope and applicability of the stainless steel bolt specifications: ASTM F593 (ASTM, 2017b), ASTM A320/A320M (ASTM, 2018b), ASTM A193/A193M (ASTM, 2020h), and ASTM A1082/A1082M (ASTM, 2016d).

Section A3.3 provides requirements for washer surface hardening treatments.

The availability of bolts, washers, and nuts made from specific alloys may be limited and should be confirmed prior to specification. Some manufacturers regularly make short runs of specialized products and higher alloyed fasteners as required, but it may be more expedient and cost effective to use off-the-shelf higher alloyed or larger fasteners. When the manufacture of specialized higher alloyed fasteners is required, it can often be arranged by the service center supplying the other components.

Nonferrous nickel alloy bolts are suitable for use in very corrosive environments where there is a need for higher strength than that possible with duplex stainless steel bolting assemblies. Note that precipitation hardening stainless steels [e.g., S17400 (17-4 PH)] are not recommended for use in corrosive environments. If the surface hardness of nonferrous nickel alloy bolts is higher than that of austenitic and duplex bolts, they are also used as a measure to avoid galling in situations where surface hardening or lubrication is not possible.

There are no specifications for nonferrous nickel alloy washers. The washers should be made from the sheet or strip standard that conforms in alloy composition to the bolt material. These standards are ASTM B443 (N06625) (ASTM, 2019h), ASTM B463 (N08020) (ASTM, 2016e), ASTM B575 (N10276, N06022, etc.) (ASTM, 2017d), ASTM B625 (N08904) (ASTM, 2017e), ASTM B670 (N07718) (ASTM, 2018c), ASTM B688 (ASTM, 2018d) or ASTM A240/A240M (N08367), and ASTM B872 (N09925, N07725, etc.) (ASTM, 2019i).

5. Surface Hardening

The metallurgical process for hardening stainless steels is very different from hardening other steel alloys. Surface hardening techniques for other steel alloys are detrimental to the corrosion resistance of stainless steels and ineffective.

Appropriate local hardening techniques may be different for the various alloys and families of stainless steels. In circumstances where the design requires specific hardening requirements (depth of hardening, type of hardening, etc.), the engineer should discuss the requirements with potential manufacturers prior to establishing the project specifications. Testing the performance and corrosion resistance of the hardened surface should be considered in critical applications if existing data are not available. Further information related to hardening can be found in Euro Inox (2015) and Davis (2002).

A4. MINIMUM ASSESSMENT REQUIREMENTS FOR SPECIFYING ALLOY CORROSION RESISTANCE

1. Galvanic Corrosion

If dissimilar metals will be in direct contact in any location, the design should be assessed to determine the potential for galvanic corrosion. Stainless steel is typically the most noble of the metals being joined and the other metals joined to it could corrode at higher rates than would be expected for the service environment if there is inadequate separation.

This assessment should be based on the relative galvanic potentials of the metals; relative surface area of the bare metals that are in direct contact; presence of moisture on a regular basis (e.g., humidity, fog, condensation, rain, immersion); and the presence of pollution or chloride salts that can accelerate galvanic corrosion. ASTM G71 (ASTM, 2019e) or ASTM G82 (ASTM, 2014a) testing may be used to help define the magnitude of the problem.

If isolation is required, the engineer should detail the means of isolation, including any coatings to be applied to separate metal sections, welded joints, or separation or sealing of bimetallic mechanically fastened joints. See AISC Design Guide 27 for more information. Water shedding atmospheric and immersed applications typically have different separation requirements.

Interior applications, such as schools, offices, warehouses, or residences, with typical temperature and humidity for human occupancy are generally not susceptible to galvanic corrosion.

2. Chloride Stress Corrosion Cracking

For stainless steel members that are highly utilized (i.e., have demands near their capacity, locally or throughout the section) and are exposed to chloride salts or chloramines, the service environment, including variations in exposure, and potential for stress corrosion cracking should be carefully considered and assessed.

For service environments where chloride stress corrosion cracking of structural members is a concern, duplex S32205 or an alloy(s) with greater resistance to chloride stress corrosion cracking should be used, unless it is demonstrated that a less corrosion-resistant alloy is acceptable.

The User Note in this Specification section points out some specific environments wherein highly utilized, heavily worked, cold-formed stainless steel structural members may be specifically prone to chloride stress corrosion cracking, but this should not be considered a complete list. See AISC Design Guide 27 for more information.

3. Crevice Corrosion

All metals are subject to crevice corrosion, which can be accelerated by galvanic corrosion when there are dissimilar metal combinations. Locations that are shielded or occluded can have low localized oxygen concentrations that are too low to maintain a passive film. These areas also frequently accumulate concentrated potentially corrosive solutions.

Examples of tight crevices include, but are not limited to, areas under washers or bolt heads, bolt or pipe fitting threads, loose gaskets or washer edges, intermittent welding, surface deposits produced by microbiological organisms, sediments or settled solids, dust, leaves, and industrial processing byproducts. Crevice corrosion is a particular concern in aqueous solutions.

A stainless steel alloy(s) with suitable crevice corrosion resistance for the service environment should be selected. The tightness of the crevice and exposure conditions determine alloy suitability. Figure C-A3.1 was developed based on research in seawater; it is critical to note that high concentrations of chlorides are not limited to applications immersed in seawater and can occur in desalination, some industrial and food processing environments, and locations with higher exposure to coastal and deicing salts, among other environments.

4. Microbiological Corrosion

Microbiologically influenced corrosion (MIC) is a form of corrosion caused by biological deposits that create a localized environment significantly different from and more corrosive than the surrounding water environment. Corrosion of metals by microorganisms in water or moist environments is a well-known problem in applications where stagnant and low flow water conditions exist. Water that is high in dissolved and suspended solids and organic material is a particular concern. These conditions allow microbes to attach to metal surfaces creating a biofilm. When these conditions exist, a stainless steel alloy that would otherwise do well in the service environment can corrode.

5. Corrosion Caused by Immersion or Splashing by Liquids

When stainless steel will be immersed in or splashed by liquids on an ongoing or regular basis, evaluation of the corrosivity of the fluid is critical. Some of the most common factors used to determine the corrosivity of the liquid are listed in Table C-A3.1. In natural liquids, the susceptibility to MIC also should be assessed.

Locations exposed to splashing by liquids or those that go through other types of regular wetting and drying cycles are typically more corrosive than those that are fully immersed. Any corrosive substances in the liquid concentrate as they dry and regular dampening makes them actively corrosive and particularly aggressive.

In water with higher levels of chlorides, usually a stainless steel with corrosion resistance that is equal to or greater than duplex S32205 is necessary; these include salt or brackish water, run-off from deiced streets, and other high chloride environments. In more corrosive fluids, the use of 6% molybdenum austenitic stainless steels (S31254, N08367, and N08926), super duplex seawater stainless steels (S32750 and S32760), or alloys with equivalent or greater corrosion resistance is common.

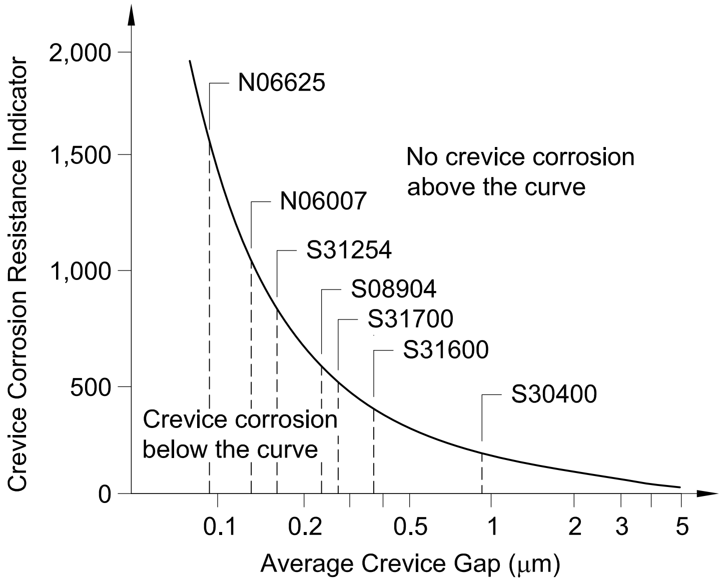


Fig. C-A3.1. Approximate corrosion resistance limits for various gap (i.e., crevice) thicknesses in ambient sea water for some common corrosion-resistant materials (ISSF, 2002).

CHAPTER B
DESIGN REQUIREMENTS

B1. GENERAL PROVISIONS

This Specification is meant to be primarily applicable to the common types of building frames with gravity loads carried by beams and girders and lateral loads carried by moment frames, braced frames, or shear walls. However, there are many unusual buildings or building-like structures for which this Specification is also applicable. Rather than attempt to establish the purview of the Specification with an exhaustive classification of construction types, Section B1 requires that the design of members and their connections be consistent with the intended use of the structure and the assumptions made in the analysis of the structure.

B2. LOADS AND LOAD COMBINATIONS

The loads, load combinations, and nominal loads for use with this Specification are given in the applicable building code. In the absence of an applicable specific local, regional, or national building code, the loads (e.g., D, L, L_r, S, R, W, and E), load factors, load combinations, and nominal loads (numeric values for D, L, and other loads) are as specified in *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, ASCE/SEI 7 (ASCE, 2016), hereafter referred to as ASCE/SEI 7.

This Specification is based on strength limit states that apply to structural steel design in general. The Specification permits design for strength using either load and resistance factor design (LRFD) or allowable strength design (ASD). Information regarding the load combinations included in ASCE/SEI 7, which are applicable to design using the LRFD and ASD approach, can be found in the Commentary to the 2016 AISC *Specification.*

B3. DESIGN BASIS

The design provisions in this Specification are intended to result in an acceptably small probability of exceeding a limit state by stipulating the combination of load factors, resistance or safety factors, nominal loads, and nominal strengths consistent with the design assumptions.

Two kinds of limit states apply to structures: (a) strength limit states, which define safety against local or overall failure conditions during the intended life of the structure, and (b) serviceability limit states, which define functional requirements. This Specification, like other structural design codes, focuses primarily on strength limit states because of overriding considerations of public safety. This does not mean that limit states of serviceability (see Chapter L) are not important to the designer, who must provide for functional performance and economy of design. However, serviceability considerations permit more exercise of judgment on the part of the designer.

The relevant information for each subsection of Section B3, excluding subsections B3.1, B3.2, B3.3, B3.10, and B3.11, can be found in the Commentary to the 2016 AISC *Specification*. Information related to the other subsections is given below.

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

Design for strength by LRFD is performed in accordance with Equation B3-1. The left side of Equation B3-1, R_u, represents the required strength computed by structural analysis based on load combinations stipulated in ASCE/SEI 7, Section 2.3 (or their equivalent), while the right side, ϕR_n, represents the limiting structural resistance, or design strength, provided by the member or element.

The LRFD provisions in this Specification are based on first-order probabilistic models of loads and resistance similar to those used in the 2016 AISC *Specification*. In cases where not enough data were available to perform a reliability analysis, the provisions given in the 2016 AISC *Specification* were adopted as long as they were considered to provide a conservative solution.

The reliability index used in the determination of the resistance factors was calculated using Equation C-B3-1:

$$\beta = \frac{\ln\left(R_m / Q_m\right)}{\sqrt{V_R^2 + V_Q^2}} \tag{C-B3-1}$$

where

Q_m = mean value of the load effect
R_m = mean value of the resistance
V_Q = coefficient of variation of the load effect, Q
V_R = coefficient of variation of the resistance, R

In accordance with the assumptions made in the development of the LRFD approach used in the 2016 AISC *Specification*, the target reliability index adopted in this Specification for members was $\beta = 2.6$ and for connections was $\beta = 4.0$ (Bartlett et al., 2003). The reliability analysis assumed a dead-to-live load ratio, $D/L = 3$, while for the basic combination of dead plus live load, a dead load factor of 1.2 and a live load factor of 1.6 were also used.

Q_m and V_Q were calculated from the following equations given in Ellingwood et al. (1980). These expressions were also used by Lin et al. (1992):

$$Q_m = c(D_m + L_m) \tag{C-B3-2}$$

$$V_Q = \frac{\sqrt{\left(D_m V_D\right)^2 + \left(L_m V_L\right)^2}}{\left(D_m + L_m\right)} \tag{C-B3-3}$$

where

c = influence coefficient that transfers load intensities to load effects

The following values for the parameters were adopted in the calculation of the resistance factors: $D_m = 1.05D_n$, $V_D = 0.1$, $L_m/L = 1.0$, and $V_L = 0.25$. The subscripts m and n refer to mean and nominal, respectively. Assuming a dead load-to-live load ratio, $D/L = 3$, gives $V_Q = 0.19$ and $Q_m = 1.33cL_m$.

The mean resistance of a structural member, R_m, is defined as follows:

$$R_m = R_n M_m F_m P_m \tag{C-B3-4}$$

where

R_n = nominal resistance of the structural elements

M_m, F_m, P_m = mean values of the random variables reflecting the uncertainties in material properties (i.e., F_y, F_u, etc.), the geometry of the cross section (i.e., A, t, L, etc.), and the design assumptions, respectively

M, known as the material factor, is taken as the ratio of the actual measured value of a mechanical property to the minimum specified value of that property given in the relevant ASTM standard. Similarly, F, known as the fabrication factor, is taken as the ratio of the actual measured value of that geometrical property to the nominal value of that property. P, known as the professional factor, is taken as the ratio of the measured failure load to the failure mode predicted from the design provision.

The coefficient of variation of the resistance, V_R, is calculated as the square-root-sum-of-squares of the material, fabrication, and design model uncertainty coefficients of variation:

$$V_R = \sqrt{V_M^2 + V_F^2 + V_P^2} \tag{C-B3-5}$$

The mean values for the material factors, M_m, and corresponding coefficient of variations, V_m, used to calculate the resistance factors are listed in Table C-B3.1. These factors were derived from a statistical study carried out at the Steel Construction Institute in the UK, where a large amount of tensile test results on austenitic and duplex stainless steel sheets and plates with thicknesses ranging from 0.02 in. (0.4 mm) to 2 in. (50 mm) were collected and analyzed (Meza et al., 2020). The mean value for the fabrication factors, F_m, and corresponding coefficient of variation for stainless steel members and bolted connections, V_F, were assumed to be the same as those used in the *Specification for the Design of Cold-Formed Stainless Steel Structural Members* (ASCE, 2021), and in the development of the AISC LRFD criteria used in the 2016 AISC *Specification*. These are F_m = 1.00 and V_F = 0.05. For welded connections, the values of F_m = 1.07 and V_F = 0.14 proposed by Li et al. (2007), which were obtained after analyzing more than 1,700 carbon steel welded connection specimens reported by several research groups, were adopted.

Knowing the values of these statistical parameters, the resistance factors, ϕ, were calculated using Equation C-B3-6:

$$\phi = \frac{1.481 M_m F_m P_m}{\exp\left(\beta\sqrt{V_R^2 + V_Q^2}\right)} \tag{C-B3-6}$$

TABLE C-B3.1
Material Factors

Alloy Family	Yield Stress, F_y		Tensile Strength, F_u	
	M_m	V_M	M_m	V_M
Austenitic	1.25	0.08	1.10	0.04
Duplex	1.10	0.04	1.10	0.04

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

Design for strength by ASD is performed in accordance with Equation B3-2. The ASD method provided in the Specification recognizes that the controlling modes of failure are the same for structures designed by ASD and LRFD. Thus, the nominal strength that forms the foundation of LRFD is the same nominal strength that provides the foundation for ASD. When considering available strength, the only difference between the two methods is the resistance factor in LRFD, ϕ, and the safety factor in ASD, Ω.

Throughout this Specification, the values of Ω were obtained from the values of ϕ. The relationship between Ω and ϕ in this Specification is the same as in the 2016 AISC *Specification*, and can be determined by equating the designs for the two methods at a dead-to-live load ratio, $D/L = 3$, which is the ratio used in the development of the LRFD resistance factors. Using the live plus dead load combinations, with $L = 3D$, yields the following relationships.

For design according to Section B3.1 (LRFD)

$$\phi R_n = 1.2D + 1.6L = 1.2D + 1.6(3D) = 6D \tag{C-B3-7}$$

$$R_n = \frac{6D}{\phi}$$

For design according to Section B3.2 (ASD)

$$\frac{R_n}{\Omega} = D + L = D + 3D = 4D \tag{C-B3-8}$$

$$R_n = \Omega(4D)$$

Equating R_n from the LRFD and ASD formulations and solving for Ω yields

$$\Omega = \frac{6D}{\phi}\left(\frac{1}{4D}\right) = \frac{1.5}{\phi} \tag{C-B3-9}$$

3. Required Strength

This Specification permits the use of elastic or inelastic structural analysis. Generally, design is performed by elastic analysis. Provisions for inelastic analysis are given in Appendix 1. The required strength is determined by the appropriate methods of structural analysis.

In some circumstances, as in the proportioning of stability bracing members that carry no calculated forces (see, e.g., Appendix 6), the required strength is explicitly stated in this Specification.

The moment redistribution provisions included in the 2016 AISC *Specification* are not included in this Specification because stainless steel structures are less well represented by plastic hinge based concepts and are more accurately and effectively designed using the provisions of Appendix 1, Section 1.3.

10. Design for Ponding

As used in this Specification, ponding refers to the retention of water due solely to the deflection of roof framing under all loads (including dead loads) present at the onset of ponding and the subsequent accumulation of rainwater and snowmelt. The amount of accumulated water is dependent on the stiffness of the framing. Unbounded incremental deflections due to the incremental increase in retained water can result in the collapse of the roof. The problem becomes catastrophic when more water causes more deflection, resulting in a greater opportunity for more water to accumulate until the roof collapses.

This Specification requires that design for ponding be considered if water is impounded on the roof, irrespective of roof slope. Camber and deflections due to loads acting concurrently with rain loads must be considered in establishing the initial conditions.

Determination of ponding stability is typically done by structural analysis where the rain loads are increased by the incremental deflections of the structural framing system to the accumulated rain water, assuming the primary roof drains are blocked.

Due to the nonlinear characteristics of stainless steels, it is necessary to use the secant modulus, as opposed to the modulus of elasticity for estimating deflections. Compared to carbon steels, greater deflections will occur in austenitic stainless steel beams at high strains, increasing the occurrence of ponding and the weight of ponded water. Compared to carbon steels, the increase in deflection in duplex stainless steels is less significant.

11. Design for Fatigue

This section provides the charging language for Appendix 3 on design for fatigue. Although fatigue need not be considered for seismic effects, it is not stated in Section B3.11 because seismic design is outside the scope of this Specification.

B4. MEMBER PROPERTIES

The User Note in this section is to acknowledge that the engineer may not always know with certainty whether the member being designed will consist of a hot-rolled section or a built-up section. The presence of the fillet radius in a hot-rolled section leads to slightly larger cross-sectional properties compared to an equivalent section without a fillet radius. For this reason, in most cases, assuming that the section is built up will lead to slightly conservative strength predictions. However, for hot-rolled sections with tapered flanges, such as S- or C-shapes, assuming that the section is built up may lead to an overestimation of the strength for limit states involving bending about the minor axis or torsion, such as minor-axis flexural buckling or lateral-torsional buckling. This is because in a section with tapered flanges, the thickness at the tip of the flanges is thinner than at the web-flange junction, which leads to a reduction of the minor-axis properties and torsional properties when compared to an equivalent section with constant flange thickness. It should be noted, however, that in most cases, the difference in strength will be small enough so that whether the section is a built-up section or a hot-rolled section will not affect the choice of the section size.

1. Classification of Sections for Local Buckling

The limiting width-to-thickness ratios listed in Table B4.1 are based on the susceptibility of the cross section to local buckling relative to the attainment of the 0.2% offset yield strength. These limits were derived by calibration against experimental data on austenitic and duplex stainless steel members and differ from those given in the 2016 AISC *Specification* for carbon steel due to the nonlinear stress-strain characteristics of stainless steel.

Limiting Width-to-Thickness Ratios for Compression Elements in Members Subject to Axial Compression. Compression members containing any elements with width-to-thickness ratios greater than λ_r provided in Table B4.1a are designated as slender and are subject to the local buckling reductions detailed in Section E7. Nonslender compression members (all elements having width-to-thickness ratio $\leq \lambda_r$) are not subject to local buckling reductions.

The limiting width-to-thickness ratios, λ_r, for the cross-section elements of compression members other than round HSS are based on the effective width curve given by Equations E7-2 and E7-3, and correspond to the maximum ratio for which there is no reduction in the width of the cross-section element. For round HSS, λ_r corresponds to the Class 3 limit for round HSS in Eurocode 3 Part 1-4 (CEN, 2020).

Limiting Width-to-Thickness Ratios for Compression Elements in Members Subject to Flexure. Flexural members containing compression elements, all with width-to-thickness ratios less than or equal to λ_p as provided in Table B4.1b, are designated as compact. Compact sections are capable of developing the plastic moment, M_p, before the onset of local buckling. Flexural members containing any compression element with width-to-thickness ratios greater than λ_p, but still with all compression elements having width-to-thickness ratios less than or equal to λ_r, are designated as noncompact. Noncompact sections can develop the yield moment, M_y, before local buckling occurs, but will not be able to attain the plastic moment. Flexural members containing any compression elements with width-to-thickness ratios greater than λ_r are designated as slender. Slender-element sections have one or more compression elements that will buckle before the yield stress is achieved. Noncompact and slender-element sections are subject to flange local buckling and/or web local buckling reductions as provided in Chapter F and summarized in Table User Note F1.1.

The limiting width-to-thickness ratios, λ_p, for flexural members given in Table B4.1b were adopted based on research from Gardner and Theofanous (2008), and they are broadly equivalent to the Class 2 limits used in the Eurocode 3 Part 1-4 (CEN, 2020) for the relevant stress distribution in the element. An exception to this equivalence are the limiting width-to-thickness ratios for flanges of I-shaped sections and channels in flexure about the minor axis, for which the limits were conservatively chosen to be the same as those for flanges subject to a uniform stress distribution, following the recommendations of Bu and Gardner (2018). The limiting width-to-thickness ratios, λ_r, for flexural members other than round HSS were derived from the effective width curve given by Equation F3-3, while for round HSS, λ_r corresponds to the Class 3 limit for round HSS used in Eurocode 3 Part 1-4 (CEN, 2020).

2. Design Thickness

It is common for suppliers to order and deliver material that is close to minimum tolerances, and typically fabricators and design engineers have no control over, or minimal involvement, in this process.

ASTM thickness, size, area, and weight tolerances vary for different stainless steel products. Some ASTM product specifications give size tolerances but no thickness tolerances, and some give weight tolerances, but no thickness tolerances. No thickness, size, or weight tolerances are given for stainless steel hot-rolled I-shaped sections.

The 2016 AISC *Specification* imposes a design wall thickness of 0.93 times the nominal wall thickness for HSS in accordance with ASTM A500/A500M (ASTM, 2013), due to the large thickness tolerance permitted for products in that specification (−10%). The derivation of the 0.93 factor was based on a survey carried out by the Steel Tube Institute of North America in the 1990s, and the details are not in the public domain.

The design thicknesses used in this Specification are based on an analysis of the thickness, size, and weight tolerances in all the relevant stainless steel product specifications [ASTM A480/A480M (ASTM, 2020a), ASTM A484/A484M (ASTM, 2020c), ASTM A554 (ASTM, 2016b), ASTM A312/A312M (ASTM, 2019b), and ASTM A999/A999M (ASTM, 2018g)]. Assuming that the fabrication factor for structural members is only dependent on the thickness of the structural shape, adopting a design thickness different from the nominal thickness has the effect of shifting the mean value of the fabrication factor, as shown by Equation C-B4-1:

$$F_{m,des} = \frac{t_{des}}{t_{nom}} F_m \tag{C-B4-1}$$

where

F_m = mean value of the fabrication factor
$F_{m,des}$ = mean value of the fabrication factor associated with the design thickness
t_{des} = design thickness, in. (mm)
t_{nom} = nominal thickness, in. (mm)

For all products where the thickness tolerance (or approximately equivalent tolerance, where no thickness tolerance is given) exceeds −5%, the design thickness was set to 0.95 times the nominal thickness, as summarized in Table C-B4.1. For $F_{m,des} = 0.95$ and $V_F = 0.05$, the reliability analysis assumes that the probability of the actual thickness being less than 0.90 times the nominal thickness of the structural shapes fabricated according to the relevant ASTM standard is 16%, which was considered to be acceptable.

Whenever a customer specifies that a product should comply with a thickness tolerance less than 5%, it is permitted to use the nominal thickness for the design thickness, irrespective of the thickness.

Published values for the section properties of hot-rolled carbon steel sections in which the thickness of the flange or web are less than or equal to 3⁄16 in. (5 mm), should not be used for stainless steel hot-rolled sections.

3. Strength Increase in Stainless Steel HSS from Cold Forming

Cold work is known to increase the yield stress of cold-formed stainless steel (Karren, 1967; van den Berg and van der Merwe, 1992; Ashraf et al., 2005; Cruise and Gardner, 2008). While in press-braked sections the increased strength is mainly localized around the corner regions, in cold-rolled sections, strength increases arise in both the corner regions and the flat portions of the section. The increase in strength also depends on the type of material. Thus, cold-rolled austenitic stainless steel sections may exhibit an increase in the yield strength of up to 48%.

The predictive model given in this section determines the average tensile 0.2% proof strength of cold-formed sections, and involves the determination of the cold work induced plastic strains in the relevant parts of the section followed by the evaluation of the corresponding stress from the stress-strain response of the unformed sheet material. The method is based on research carried out by Afshan et al. (2013) and Rossi et al. (2013).

The User Note in this section is to warn the designer that the average yield strength should not be used to verify the strength of members at cross sections where the member is welded to other parts of the structure. This is due to the annealing effect of the weld in the heat affected zone that may significantly reduce or even completely remove the increase in strength achieved from cold work during the fabrication of the member. Because members are likely to be welded to other parts of the structure at their ends, and member buckling leads to localized failure away from the ends, in most cases it will still be possible to use the average yield strength, $F_{y,avg}$, to determine the strength of the member for member buckling limit states. However, if welds are present near the midpoint of the member, it is recommended that the design is based on the specified minimum yield stress, F_y.

TABLE C-B4.1
Design Thickness for Different Product Forms

Built-Up, Hot-Rolled, and Flat Bar Sections	
For thickness ≤ 3⁄16 in. (5 mm)	Design thickness = 0.95 × nominal thickness
For thickness > 3⁄16 in. (5 mm)	Design thickness = nominal thickness
HSS and Pipe	
For all thicknesses	Design thickness = 0.95 × nominal thickness

4. Gross and Net Area Determination

4a. Gross Area

Gross area is the total area of the cross section without deductions for holes or ineffective portions of elements subject to local buckling.

4b. Net Area

The net area is based on net width and load transfer at a particular chain. Because of possible damage around a hole during drilling or punching operations, 1⁄16 in. (2 mm) is added to the nominal hole diameter when computing the net area.

B5. FABRICATION AND ERECTION

Section B5 provides the charging language for Chapter M on fabrication and erection.

B6. QUALITY CONTROL AND QUALITY ASSURANCE

Section B6 provides the charging language for Chapter N on quality control and quality assurance.

B7. EVALUATION OF EXISTING STRUCTURES

Section B7 provides the charging language for Appendix 5 on the evaluation of existing structures.

CHAPTER C

DESIGN FOR STABILITY

Chapter C addresses the stability design requirements for stainless steel buildings and other structures. It is based upon the direct analysis method, which can be used in all cases.

The provisions are based on the 2016 AISC *Specification*, but with the modifications described herein. Further information can be found in the Commentary to the 2016 AISC *Specification*.

C2. CALCULATION OF REQUIRED STRENGTHS

The provisions in the 2016 AISC *Specification* were modified where necessary to account for the differences in material behavior between stainless steel and carbon steel and the resulting influence on structural behavior. Stainless steel exhibits rounded stress-strain behavior and the early onset of plasticity results in increased degradation of stiffness and enhanced second-order effects (Walport et al., 2019a).

3. Adjustments to Stiffness

For design by second-order elastic analysis, two stiffness reduction factors are defined: (1) a general stiffness reduction factor, τ_g, to be applied to all member stiffnesses to account for the development and spread of plasticity, and (2) τ_b, to account for the additional reduction in flexural stiffness due to the effects of yielding and residual stresses of heavily loaded compression members. Additionally, for slender members, the τ_g factor results in a system required strength equal to τ_g times the elastic stability limit, which is equivalent to the margin of safety provided by the column buckling curves (AISC-SSRC, 2003).

The adopted stiffness reduction factor, τ_b, for stainless steel, accounting for the combined effects of material nonlinearity and residual stresses, was derived directly from the Ramberg-Osgood expression (Appendix 7, Equation A-7-1a). The stiffness reduction factor, τ_b, was taken as the ratio of the tangent modulus, E_t, to the modulus of elasticity, E, where $E_t = df/d\varepsilon$ and $f = P_r/A$, where P_r is the required axial compressive strength and A is the cross-sectional area. To allow for the influence of residual stresses, the strain hardening coefficient, n, was modified to an effective strain hardening coefficient, n_{eff}, through calibration against finite element analysis results for the tangent flexural stiffness obtained from a W8×31 cross section divided into 1,440 monitoring areas, subjected to pure axial compression (Gardner and Walport, 2021).

The differing values of n_{eff} for the different buckling axes reflect the fact that the flexural stiffness is reduced more severely for the minor axis than the major axis due to the more detrimental influence of the compressive residual stresses at the flange tips. In the case of rectangular hollow structural sections (HSS), since the residual stresses are very small, the effective strain hardening coefficient, n_{eff}, is taken equal to n. To retain the same demarcation between cross sections as the flexural buckling curves

(Table E3.1), the stiffness reduction function for welded box sections and round HSS is taken equal to that for I-sections buckling about the major axis.

Because n_{eff} is a function of n, the varying degrees of roundedness of the stress-strain behavior, as shown in Figure C-C2.1 for the typical grades of stainless steel (austenitic grade 301 and duplex grade S32101) is reflected in τ_b. Alongside the proposed stiffness reduction factors, the carbon steel stiffness reduction factor given in the 2016 AISC *Specification*, is also presented. It can be seen that, unlike for carbon steel, the stiffness reduction for stainless steel commences from the onset of loading. This reflects the rounded stress-strain response of stainless steel, exacerbated by the influence of the residual stresses. The greatest reduction at low to moderate axial load levels occurs for austenitic stainless steel due to the low limit of proportionality and the low value of the strain hardening coefficient, n, resulting in the highest degree of nonlinearity of stress-strain response among the austenitic and duplex families of stainless steel.

Application of the stiffness reduction method is inherently an iterative process; the stiffness reduction factors, τ_b, are calculated at the load level of interest and the results of the subsequent second-order analysis are only valid at that same load level. An alternative, simpler approach that avoids the need for iteration is to replace the use of τ_b factors (i.e., by setting τ_b = 1.0 for all members) with the application of additional notional loads (ANL). The enhanced notional loads are designed to account indirectly for the effect of the spread of plasticity and residual stresses on the global response of the structure. The stiffness reduction factors, τ_b, derived for stainless steel are more severe than those for carbon steel, reflecting the earlier initiation of yielding of the material. A commensurate increase in the ANL from 0.001 to 0.002 of the total factored gravity load applied at each story of structural frames is therefore required (Gardner and Walport, 2021).

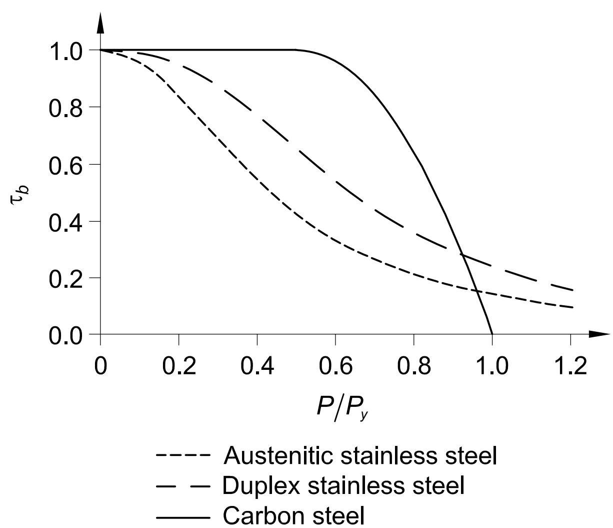


Fig. C-C2.1. Stiffness reduction factor, τ_b, for typical austenitic and duplex grades of stainless steel for I-shaped sections buckling about the minor axis.

The τ_g values given in Table C2.1 were derived by Gardner and Walport (2021) based on benchmark results from nonlinear finite element analysis on pin-ended stainless steel columns, beams, and beam-columns, as well as sample frames. A value of 0.70 was found to be sufficient to account for the influence of plasticity.

CHAPTER D

DESIGN OF MEMBERS FOR TENSION

The provisions of Chapter D do not account for eccentricities between the lines of action of connected assemblies.

This Specification does not include provisions for eyebars due to a lack of test data of this product form.

D1. SLENDERNESS LIMITATIONS

The advisory upper limit on slenderness in the User Note is based on professional judgment and practical considerations of economics, ease of handling, and care required so as to minimize inadvertent damage during fabrication, transport, and erection. This slenderness limit is not essential to the structural integrity of tension members; it merely assures a degree of stiffness such that undesirable lateral movement ("slapping" or vibration) will be unlikely. Out-of-straightness within reasonable tolerances does not affect the strength of tension members. Applied tension tends to reduce, whereas compression tends to amplify, out-of-straightness.

For single angles, the radius of gyration about the z-axis produces the maximum L/r and, except for very unusual support conditions, the maximum effective slenderness ratio.

D2. TENSILE STRENGTH

Because of strain hardening, a ductile stainless steel bar loaded in axial tension can resist, without rupture, a force greater than the product of its gross area and its specified minimum yield stress. However, excessive elongation of a tension member due to uncontrolled yielding of its gross area not only marks the limit of its usefulness, but can precipitate failure of the structural system of which it is a part. On the other hand, depending upon the reduction of area and other mechanical properties of the steel, the member can fail by rupture of the net area at a load smaller than required to yield the gross area. Hence, general yielding of the gross area and rupture of the net area both constitute limit states.

This Specification adopted the provisions included in the 2016 AISC *Specification* without modification for determining the design tensile strength, $\phi_t P_n$, and the allowable tensile strength, P_n/Ω_t, of austenitic and duplex stainless steel tension members. For the limit state of tensile yielding in the gross section, the ϕ and Ω values for precipitation hardening stainless steel bolts are reduced by 10% from the values for austenitic and duplex stainless steel. This is due to the lack of sufficient data to enable a reliability analysis to be carried out. Additionally, because stainless steel is twice as ductile and also exhibits strong strain hardening (F_u/F_y is approximately 2.2 for annealed austenitic stainless steels), larger deformations are expected with stainless steel than with carbon steel. For this reason, for structures that are sensitive to deformations, a User Note was added providing an expression for calculating the average stress in the net section based on the maximum elongation the tensile member is allowed to develop without compromising the integrity of the structural system. The expression given in the User Note provides a conservative estimation of the stress for strains up to the strain at the ultimate tensile stress, ε_u. For cases in which a more accurate estimation of the stress is required, the User Note refers the designer to the material model included in Appendix 7.

The expression given in the User Note could also be used to limit the stresses in the net section due to service loading in structures that are sensitive to deformation at serviceability.

D3. EFFECTIVE NET AREA

The expressions for the calculation of the effective net area included in the 2016 AISC *Specification*, which account for the effect of the shear lag, were adopted in this Specification without modification. Information regarding the development of these expressions can be found in the Commentary to the 2016 AISC *Specification*.

D5. PIN-CONNECTED MEMBERS

Pin-connected members are occasionally used as tension members with very large dead loads. Pin-connected members are not recommended when there is sufficient variation in live loading to cause wearing of the pins in the holes. The dimensional requirements presented in Section D5.2 must be met to provide for the proper functioning of the pin.

1. Tensile Strength

The tensile strength requirements for stainless steel pin-connected members are the same as those included in the 2016 AISC *Specification*. The ϕ and Ω values used to calculate the design tensile strength, $\phi_t P_n$, and allowable tensile strength, P_n/Ω_t, are the same as those used elsewhere in this Specification for similar limit states. However, the definitions of effective net area for tension and shear are different.

The requirement for the width of the plate on each side of the pin hole, b_e, not to exceed $2t + 0.63$, in. ($2t + 16$, mm) is to prevent dishing of the plate, which can significantly reduce the ultimate tensile strength of thin plates. The upper bound limit for b_e was derived from research on carbon steel pin-connected members (Johnston, 1939). However, for the typical pin-connected members used in current practice, this type of failure does not generally govern the design.

2. Dimensional Requirements

The dimensional requirements for stainless steel pin-connected members are similar to those given in the 2016 AISC *Specification* for carbon steel pin-connected members, with the exception of the following two requirements.

In the 2016 AISC *Specification*, the width of the plate at the pin hole is required to be greater than or equal to $4t + d_{pin} + 1.26$, in. ($4t + d_{pin} + 32$, mm). In this Specification, the width of the plate at the pin hole is required to be at least $2d_{pin}$. This implies that for plates with a thickness $t > d_{pin}/4 - 0.32$, in. ($t > d_{pin}/4 - 8$, mm), this Specification permits the plate to be narrower than if designed in accordance with the 2016 AISC *Specification*. For these narrow plates, the reduction in tensile strength is taken into account by the smaller value of *be* used in the calculation of the net effective area.

In addition, this Specification requires the thickness of the plate to be large enough to avoid bearing failure of the pin. This requirement was added to account for the fact that when designing stainless steel pin-connected members, there are many cases in which the connected members are made from a high-strength stainless steel, while the pin is fabricated from an austenitic stainless steel alloy in the annealed condition with a lower strength.

CHAPTER E

DESIGN OF MEMBERS FOR COMPRESSION

E1. GENERAL PROVISIONS

This chapter addresses austenitic and duplex stainless steel members subject to axial compression.

Compared to the 2016 AISC *Specification*, this Specification does not include provisions for unequal-leg angles or slender angles in compression due to lack of experimental data on these types of stainless steel members. Built-up members assembled from plates or laced built-up members are also not covered by this Specification.

Although tees, single equal-leg angles (where the term *equal-leg* indicates that the angle has both legs of equal length and thickness), and double angles subject to axial compression can be designed in accordance with the provisions of Chapter E, no provisions are given in this Specification for these types of members when they are subjected to other types of forces in addition to compression. This does not include single equal-leg angle members without slender elements loaded in compression through one connected leg, which can be designed in accordance with Section E5 if the requirements given in that section are met.

The continuous strength method (CSM), included in Appendix 2, provides an alternative, less conservative method for determining the compressive strength of I-shapes, channels, angles, tees, HSS, and box-section members, when the following requirements are met.

For round HSS

$$\lambda_l \le 0.60$$

For all other structural sections

$$\lambda_l \le 1.60$$

and

$$\frac{L_c}{r} \le 0.63\sqrt{\frac{E}{F_y}} \quad \left(\text{or } \frac{F_y}{F_e} \le 0.04\right) \tag{C-E3-1}$$

where λ_l is the cross-section local slenderness, given by Equation C-E3-2:

$$\lambda_l = \sqrt{\frac{F_y}{F_{el}}} \tag{C-E3-2}$$

and F_{el} is the elastic local buckling stress of the full cross section under the relevant stress distribution, which can be determined using the analytical expressions given in Appendix 1, Section 1.3, of this Commentary, or using numerical methods. Alternatively, F_{el} may be conservatively determined using the expressions provided in the second User Note in Appendix 2, Section 2.3.

Compression members, other than round HSS, that satisfy the requirement for the slenderness ratio, L_c/r, given by Equation C-E3-1 are not affected by flexural, torsional, or flexural-torsional buckling, and the CSM determines their compressive strength purely based on the limit state of yielding or local buckling.

E2. EFFECTIVE LENGTH

This Specification adopted the same symbol, L_c, used in the 2016 AISC *Specification* to refer to the effective length, as opposed to the traditional expression, *KL*. The recommendation of a slenderness ratio not exceeding 200 for members designed on the basis of compression was also taken from the 2016 AISC *Specification*.

An explanation for the adoption of L_c, as well as the maximum recommended slenderness ratio can be found in the Commentary on the 2016 AISC *Specification*.

E3. FLEXURAL BUCKLING OF MEMBERS WITHOUT SLENDER ELEMENTS

Section E3 applies to compression members with all nonslender elements, as defined in Section B4.

The provisions in this Specification for determining the compressive strength of austenitic and duplex stainless steel members maintain the same format as those included in the 2016 AISC *Specification* for carbon steel. However, the shape of the flexural buckling curve for carbon steel was adjusted to account for the higher material nonlinearity and strain hardening exhibited by stainless steel. In addition to that, while the 2016 AISC *Specification* specifies only one buckling curve for carbon steel columns irrespective of their cross-section geometry or axis of buckling, due to the higher material cost of stainless steel, in this Specification three different buckling curves are specified, as shown by Table E3.1.

The flexural buckling curves for HSS, box sections, and I-shaped columns buckling about the major axis were developed by Meza et al. (2021) based on experimental and numerical data available for austenitic and duplex stainless steel columns. For I-shaped columns, this includes experimental data reported by Burgan et al. (2000), Yuan et al. (2015), and Yang et al. (2016a) and the numerical data reported by Bu and Gardner (2019). For rectangular HSS and welded box sections, the flexural buckling curve is based on the experimental and numerical data reported by Young and Liu (2003), Liu and Young (2003), Rasmussen and Hancock (1993), Gardner and Nethercot (2004b), Gardner et al. (2006), Huang and Young (2013), Theofanous and Gardner (2009), Young and Lui (2006), Yang et al. (2016b), Afshan et al. (2019), and Becque (2008), while for round HSS the data reported by Burgan et al. (2000), Rasmussen and Hancock (1993), and Buchanan et al. (2018) was considered. For I-shaped columns buckling about the minor axis, and any other cross-sectional shape not listed in Table E3.1, this Specification adopted the same buckling curve used in the *Specification for the Design of Cold-Formed Stainless Steel Structural Members* (ASCE, 2021). Despite the different levels of nonlinearity and strain hardening exhibited by austenitic and duplex stainless steel, the same flexural buckling curves were adopted for both alloy families. For rectangular and round HSS columns, although the data showed that the flexural buckling strength, relative to the squash load ($P_y = F_y A_g$), of duplex alloys is slightly higher than that of austenitic alloys, the larger material overstrength factor of austenitic alloys justified adopting the same buckling curve for both alloy families. For I-shaped columns, the reason for adopting the same buckling curve for austenitic and duplex stainless steel columns is because most of the data used to derive the flexural buckling curves were on austenitic alloys, and using the same curve for duplex stainless steel is conservative.

Figure C-E3.1 compares the strength provisions adopted in this Specification for austenitic and duplex stainless steel columns, and those included in the 2016 AISC *Specification* for carbon steel columns. The curves in Figure C-E3.1 show the relationship between the nominal compressive strength, P_n, and the column slenderness ratio, L_c/r, for the limit state of flexural buckling. For the sake of comparison, in the figure, P_n is normalized by P_y, and L_c/r is normalized by $\sqrt{E/F_y}$.

An important characteristic that differentiates the flexural buckling curves adopted in this Specification from the ones included in the 2016 AISC *Specification* is that all the flexural buckling curves in this Specification include a yield plateau for small slenderness ratios, L_c/r, defined as $\beta_0\sqrt{E/F_y}$, where β_0 is given in Table E3.1 for austenitic and duplex stainless steel columns with different cross-section geometries and axes of buckling. When the slenderness ratio is smaller than $\beta_0\sqrt{E/F_y}$, the

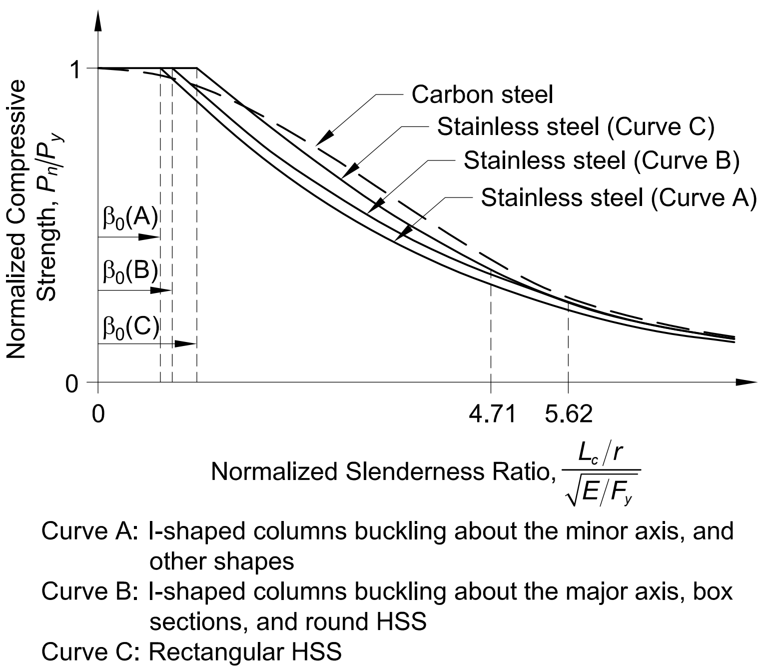


Fig. C-E3.1. Nominal compressive strength as a function of slenderness ratio for columns made of stainless steel and carbon steel.

column is considered not to be affected by flexural buckling, and its compressive strength is given by P_y. The yield plateau in the flexural buckling curves recognizes the high strength exhibited by stainless steel columns at low slenderness as a result of strain hardening, which, depending on the cross-sectional slenderness, can be well above P_y. The inclusion of a yield plateau in the flexural buckling curve of austenitic and duplex stainless steel columns also allows for compatibility with the use of the CSM given in Appendix 2 for those cases in which the compressive strength is not governed by flexural buckling.

The compressive strength of austenitic and duplex stainless steel columns with slenderness ratios ranging between $\beta_0\sqrt{E/F_y}$ and $5.62\sqrt{E/F_y}$ is governed by inelastic flexural buckling at a critical stress, F_{cr}, given by Equation E3-3. For larger slenderness ratios, the flexural buckling strength is predominantly elastic, as reflected by the use of the Euler stress, F_e, in the definition of F_{cr} given by Equation E3-4. The use of the reduction coefficient, β_2, in Equation E3-4 accounts primarily for the influence of the initial out-of-straightness of the column. However, it also accounts for some loss of stiffness experienced by austenitic and duplex stainless steel columns prior to the attainment of the critical buckling stress, F_{cr}, which is the reason the values of β_2 given in Table E3.1 are lower than the value of 0.877 used in the equations in the 2016 AISC *Specification* for carbon steel columns.

The column strength equations of Section E3 can also be used for torsional or flexural-torsional buckling (Section E4). They can also be used with a modified slenderness ratio for single equal-leg angle members (Section E5).

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

Section E4 applies to singly symmetric and certain doubly symmetric members, such as cruciform or built-up columns with all nonslender elements, as defined in Section B4.1 for elements in uniform compression. It also applies to all doubly symmetric members when the torsional unbraced length exceeds the lateral unbraced length of the member.

The equations in Section E4 for determining the torsional and flexural-torsional elastic buckling loads of columns are derived in textbooks and monographs on structural stability (Bleich, 1952; Timoshenko and Gere, 1961; Galambos, 1968; Chen and Atsuta, 1977; Galambos and Surovek, 2008; and Ziemian, 2010). Because these apply only to elastic buckling, they must be modified for inelastic buckling by the appropriate equations of Section E3. Inelasticity has a more significant impact on warping torsion than St. Venant torsion. For consideration of inelastic effects, the full elastic torsional or flexural-torsional buckling stress is conservatively used to determine F_e for use in the column equations of Section E3.

Members with lateral bracing offset from the shear center are susceptible to constrained-axis torsional buckling, which is discussed in the Commentary to the 2016 AISC *Specification*.

E5. SINGLE EQUAL-LEG ANGLE COMPRESSION MEMBERS WITHOUT SLENDER ELEMENTS

The compressive strength of single equal-leg angles is determined in accordance with Sections E3 for the limit state of flexural buckling and Section E4 for the limit state of flexural-torsional buckling. However, for the type of angles that meet the requirements of Section E5, flexural-torsional buckling is not a critical failure mode.

Section E5 also provides a simplified procedure for the design of single equal-leg angles subjected to an axial compressive load introduced through one connected leg. The angle is treated as an axially loaded member by adjusting the member slenderness. The attached leg is to be attached to a gusset plate or the projecting leg of another member by welding or by a bolted connection with at least two bolts. The equivalent slenderness expressions in this section presume significant restraint about the axis that is perpendicular to the connected leg. This leads to the angle member tending to bend and buckle primarily about the axis parallel to the attached gusset. For this reason, L/r_a is the slenderness parameter used, where the subscript, a, represents the axis parallel to the attached leg. The modified slenderness ratios indirectly account for bending in the angles due to the eccentricity of loading and for the effects of end restraint from the members to which they are attached. The equivalent slenderness expressions also presume a degree of rotational restraint. Equations E5-3 and E5-4 [Section E5(b), referred to as case (b)] assume a higher degree of rotational restraint about the axis parallel to the attached leg than do Equations E5-1 and E5-2 [Section E5(a), referred to as case (a)].

In space trusses, the web members framing in from one face typically restrain the twist of the chord at the panel points and thus provide significant restraint about the axis parallel to the attached leg for the angles under consideration. It is possible that the chords of a planar truss well restrained against twist justify use of case (b), in other words, Equations E5-3 and E5-4. Similarly, simple single-angle diagonal braces in braced frames could be considered to have enough end restraint such that case (a), in other words, Equations E5-1 and E5-2, could be employed for their design. This procedure, however, is not intended for the evaluation of the compressive strength of X-brace single angles.

E6. BUILT-UP MEMBERS

The strength provisions and dimensional requirements in this section for built-up members were adopted from the 2016 AISC *Specification* without modification. However, the scope of this section was reduced to only cover built-up members composed of two or more closely spaced stainless steel shapes interconnected at intervals using welds or fasteners.

1. Compressive Strength

For a built-up member to be effective as a structural member, the end connection must be welded or pretensioned bolted. Even so, the compressive strength will be affected by the shearing deformation of the intermediate connectors. This effect is considered by using a modified slenderness ratio for the built-up member. The expressions adopted in this Specification for calculating the modified slenderness ratio have been shown to be adequate for carbon steel (Sato and Uang, 2007). However, since the effect of the larger nonlinearity exhibited by stainless steel is already captured in the column curves given in Section E3, these expressions are considered to be also suitable for stainless steel.

2. Dimensional Requirements

Limiting the slenderness ratio of each component shape between connection fasteners or welds to three-fourths of the governing global slenderness ratio of the built-up member is to prevent flexural or flexural-torsional buckling of the individual shapes between intermediate connectors.

The requirement for the end connectors or welds is given to prevent slip between the components in contact at each end of the built-up member.

E7. MEMBERS WITH SLENDER ELEMENTS

The provisions of Section E7 address the modifications to be made when one or more elements in the compression member cross section are slender. An element is considered to be slender if its width-to-thickness ratio exceeds the limiting value, λ_r, defined in Table B4.1a. As long as the element is not slender, it can support the full yield stress without local buckling. When the cross section contains slender elements, the potential reduction in capacity due to local buckling must be accounted for.

1. Slender Element Members Excluding Round HSS

The reduction in capacity due to local buckling is considered by using the effective width method.

This Specification follows the same philosophy adopted in the 2016 AISC *Specification* for determining the effective width of cross sections with slender elements other than round HSS. However, the effective width expression was modified to account for the difference in the stress-strain characteristic of stainless steel compared to carbon steel.

The effective width expression given by Equation E7-3 makes use of the elastic buckling stress, F_{el}, determined for the relevant element of the cross section using Equation E7-4 and a plate buckling coefficient $k = 4.00$ for stiffened and $k = 0.425$ for unstiffened elements.

The applicability of the effective width expression given by Equations E7-2 and E7-3 was assessed by comparing it against experimental data on austenitic and duplex stainless steel stub columns and beam members reported by Kuwamura (2003), ECSC (2000), Gardner and Nethercot (2004a), Liu and Young (2003), Young and Liu (2003), and Gardner et al. (2006). This experimental data included a wide range of cross sections, including welded I-shaped sections and cold-formed HSS. For the cold-formed sections, the strength enhancement at the corners resulting from cold forming was measured and deducted when developing the effective width expression. For this reason, the same effective width expression (Equations E7-2 and E7-3) can be used for cold-formed square and rectangular HSS in conjunction with the average yield strength, $F_{y,avg}$, given in Section B4.3, while for other shapes it is used in conjunction with the specified minimum yield strength, F_y.

Equations E7-5 and E7-6 are used in the *Specification for the Design of Cold-Formed Stainless Steel Structural Members* (ASCE, 2021), and has been included in this Specification as an alternative method for calculating the effective width of cold-formed square and rectangular HSS. When this alternative effective width curve is used, both the effective width and the compressive strength of the member must be determined based on the specified minimum yield strength, without making use of the increase in strength from cold work.

For axially loaded channels with slender flanges and pinned restraints at the ends, the shift in the effective centroid due to local buckling may result in significant second-order moments, and therefore a reduced compressive strength. The reduction in strength is closely related to the rotational restraint provided by the supports. If the end supports (which for design may be considered as pinned) are able to provide, in practice, some rotational restraint, the reduction in strength may become very small. However, if the end rotational restraints are close to a pin, the channel should be treated as a member subject to combined loading, and designed according to Section H1.1 for combined axial force and flexure about the minor axis resulting from the shift of the effective centroid.

2. Round HSS

The expression for the effective cross-sectional area given by Equation E7-8 was obtained by adjusting the curve given in the 2016 AISC *Specification* for carbon steel round HSS. The appropriateness of this expression has been corroborated by comparing it against test data reported by Stangenberg (2000), Kuwamura (2003), Gardner and Nethercot (2004b), and Buchanan et al. (2018). These data show that the expression given by Equation E7-8 is applicable to round HSS compression members with $D/t \leq 0.28E/F_y$, which approximately corresponds to the largest slenderness of the round HSS tested. The provisions in Section E7.2 were also modified to account for the interaction between local and global buckling by multiplying the limiting slenderness, λ_r, by the ratio F_y/F_{cr}. This is similar to the way in which local-global buckling interaction is accounted for in Section E7.1 for sections other than round HSS.

More slender round HSS compression members with $D/t \leq 0.44E/F_y$ (or $\lambda_l \leq 0.60$) can be designed in accordance with the continuous strength method included in Appendix 2.

CHAPTER F
DESIGN OF MEMBERS FOR FLEXURE

Chapter F applies to austenitic and duplex stainless steel members subject to simple bending about one principal axis of the cross section. That is, the member is loaded in a plane parallel to a principal axis that passes through the shear center. Simple bending may also be attained if all load points and supports are restrained against twisting about the longitudinal axis. In all cases, the provisions of this chapter are based on the assumption that points of support for all members are restrained against rotation about their longitudinal axis.

Compared to the 2016 AISC *Specification*, this Specification does not include provisions for single angles subject to bending due to lack of experimental data on these types of stainless steel flexural members. Provisions for rectangular HSS subject to lateral-torsional buckling are also not covered by this Specification.

For all sections covered in Chapter F, the highest possible nominal flexural strength is the plastic moment, $M_n = M_p$. In order to attain M_p, the beam cross section must be compact and the member must have sufficient lateral bracing.

Compactness depends on the flange and web width-to-thickness ratios, as defined in Section B4. When these conditions are not met, the nominal flexural strength diminishes. All sections in Chapter F treat this reduction in the same way. For laterally braced beams, the plastic moment region extends over the range of width-to-thickness ratios, λ, terminating at λ_p. This is the compact condition. Beyond these limits, the nominal flexural strength reduces linearly until λ reaches λ_r. This is the range where the section is noncompact. Beyond λ_r the section is a slender-element section. For stainless steel sections, the extent of each of these three ranges is different from that of a section made of carbon steel. This is partly due to the effect of the higher nonlinearity exhibited by stainless steel, but also due to the assumptions made in the definition of the width-to-thickness limits. Figure C-F1.1 compares the three ranges for the case of welded I-shaped members made of stainless steel (either austenitic or duplex) and carbon steel for the limit state of flange local buckling. The curves in Figure C-F1.1 show the relationship between the flange width-to-thickness ratio, $b_f/2t_f$, and the nominal flexural strength, M_n. For the sake of comparison, in the figure, M_n is normalized by the plastic moment, M_p, and $b_f/2t_f$ is normalized by $\sqrt{E/F_y}$.

Different lateral-torsional buckling expressions are specified for I-shaped members and channels depending on whether they are made of austenitic or duplex stainless steel. The lateral-torsional buckling (LTB) curves for austenitic and duplex stainless steel differ from each other due to the introduction of the LTB coefficients, α_{LT}, β_{LT}, $\beta_{p,LT}$, and $\beta_{y,LT}$, which reflect the different levels of material nonlinearity exhibited by these alloy families. These coefficients were derived by Meza and Baddoo (2019) based on numerical data on austenitic (316L) and duplex (2205) stainless steel I-shaped beams subject to lateral-torsional buckling. The beams were modeled under pure bending with the ends simply supported and restrained against rotation about the longitudinal axis. The numerical models were validated against the results of experimental tests on austenitic and duplex

I-shaped beams carried out by Fortan and Rossi (2021), Fortan and Rossi (2020), Burgan et al. (2000), Stangenberg (2000), and Wang et al. (2014b). A total of 590 data points were used to develop the LTB curves adopted in this Specification, including 30 different cross-sectional geometries for each alloy, and 12 global slendernesses for each cross section.

A reliability analysis carried out using the numerical data indicated that the LTB provision could be used with a resistance factor, ϕ_b, higher than 0.90. However, since these provisions were developed based on data on I-shaped beams only, and they are also intended to be used for channels, a resistance factor of $\phi_b = 0.90$ was adopted, which is consistent with the resistance factor used with all the other strength provisions in Chapter F.

The basic relationship between the nominal flexural strength, M_n, and the unbraced length, L_b, for the limit state of lateral-torsional buckling is shown by the solid curve in Figure C-F1.2 for a compact stainless steel I-shaped beam that is simply supported and subjected to uniform bending with $C_b = 1.0$. For comparison, Figure C-F1.2 also shows the relationship between M_n and L_b for the limit state of lateral-torsional buckling given by the provisions of the 2016 AISC *Specification* for the same section but made of carbon steel.

For austenitic and duplex stainless steel I-shaped beams, there are four principal zones defined by the lengths L_p, L_y, and L_r. Equation F2-7 defines the maximum unbraced length, L_p, to reach M_p. Elastic lateral-torsional buckling will occur when the unbraced length is greater than L_r, given by Equation F2-9. Equation F2-2 defines the first part of the range of inelastic LTB as a straight line between the defined limits M_p at L_p and M_y at L_y, while the second part of the inelastic range is defined by Equation F2-3 between the limits M_y at L_y and $0.3M_y$ at L_r. The buckling strength in the elastic region is given by Equation F2-4 in combination with Equation F2-5. By contrast, the LTB curve for carbon steel is divided into three zones delimited by the lengths L_p and L_r.

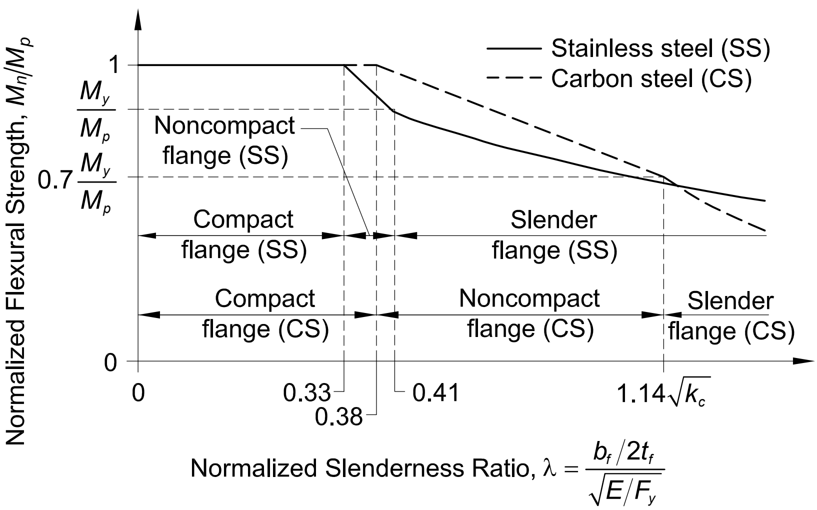


Fig. C-F1.1. Nominal flexural strength as a function of the flange width-to-thickness ratio of welded I-shapes made of stainless steel and carbon steel.

F1. GENERAL PROVISIONS

Throughout Chapter F, the resistance factor and the safety factor remain unchanged, regardless of the controlling limit state.

In addition, the provisions in this chapter require that all supports for flexural members be restrained against rotation about the longitudinal axis. Although there are provisions for members unbraced along their length, under no circumstances can the supports remain unrestrained torsionally.

The provisions for determining the lateral-torsional modification factor, C_b, for non-uniform moment diagrams when both ends of the segment are braced were taken from the 2016 AISC *Specification* without modification. Further information regarding the development of these provisions can be found in the Commentary to the 2016 AISC *Specification*.

The continuous strength method (CSM), included in Appendix 2, provides an alternative, less conservative method for determining the flexural strength of laterally restrained I-shaped, channels, angles, tees, HSS, and box-section members, when the following requirements are met.

For round HSS

$$\lambda_l \leq 0.60$$

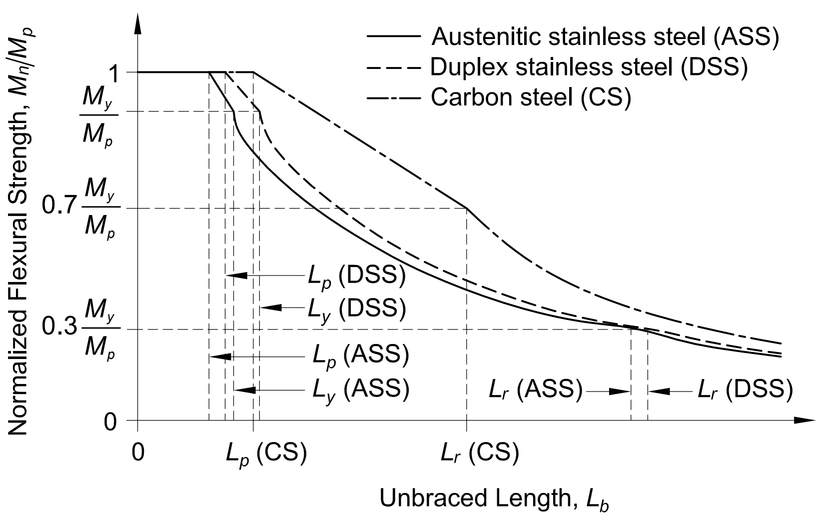


Fig. C-F1.2. Nominal flexural strength as a function of unbraced length of I-shapes made of stainless steel and carbon steel.

For all other structural sections

$$\lambda_l \leq 1.60$$

and

$$L_b \leq 0.50 L_p \tag{C-F1-1}$$

where L_b is the length between points that are either braced against lateral displacement of the compression flange or braced against twist of the cross section; L_p is the limiting laterally unbraced length for the limit state of yielding given by Equation F2-7; λ_l is the cross-section local slenderness; given by $\sqrt{F_y / F_{el}}$; and F_{el} is the elastic local buckling stress of the full cross section under the relevant stress distribution, which can be determined using the analytical expressions given in Appendix 1, Section 1.3, of this Commentary, or using numerical methods. Alternatively, F_{el} may be conservatively determined using the expressions provided in the second User Note in Appendix 2, Section 2.3.

Flexural members, other than round HSS, that satisfied the requirement for the unbraced length, L_b, given by Equation C-F1-1 are not affected by lateral-torsional buckling, and the CSM determines their flexural strength purely based on the limit state of yielding or local buckling.

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

Section F2 applies to members with compact I-shaped or channel cross sections subject to bending about their major axis; hence, the only limit states to consider are yielding (plastic moment capacity) and lateral-torsional buckling.

The expressions in this section for the limit state of yielding of austenitic and duplex stainless steel sections with compact webs and compact flanges (Equation F2-1) are the same as those in the 2016 AISC *Specification*. However, the limiting laterally unbraced length for the limit state of yielding, L_p, was reduced based on numerical data on compact I-shaped beams made of austenitic and duplex stainless steel generated by Fortan and Rossi (2021). Different L_p values are given for austenitic and duplex stainless steel beams, which reflect the effect the different levels of material nonlinearity exhibited by austenitic and duplex stainless steel have on the global stability of the beam. Another difference in the definition of L_p with respect to the 2016 AISC *Specification* is that in this Specification, L_p is defined as a function of r_{ts} (or the radius of gyration of the compression flange plus one-sixth of the web, r_t) for all types of I-shaped members, while in the 2016 AISC *Specification*, L_p is given as a function of r_y for I-shaped members with compact and noncompact webs and as a function of r_t for I-shaped members with slender webs. Defining L_p as a function of r_{ts} (or r_t) for all types of I-sections avoids a discontinuity in the LTB strength as the web transitions from a noncompact to a slender element, as discussed by White (2008).

In the 2016 AISC *Specification*, L_r is defined as the unbraced length beyond which geometric imperfections and yielding have virtually no effect on the LTB strength. For carbon steel I-shaped members and channels, when the unbraced length is equal to L_r, the maximum stress in the compression flange due to lateral-torsional buckling is equal to $0.7F_y$. Members with an unbraced length shorter than L_r fall in the inelastic region, where the effect of geometric imperfections and yielding results in a significant reduction of the LTB strength. The characteristically low proportionality limit in stainless steels means that the definition of L_r used in the 2016 AISC *Specification* cannot be rigorously adopted for stainless steel members, as this would result in limiting unbraced lengths, L_r, significantly larger than the unbraced lengths, L_b, which are likely to be used in practice. For this reason, in this Specification, the elastic LTB strength is reduced using the elastic lateral-torsional buckling reduction coefficient, β_{LT}, which was determined based on numerical data on austenitic and duplex stainless steel I-shaped beams, and the limiting unbraced length, L_r, is defined as the length at which the reduced elastic LTB strength given by Equation F2-4 (with F_{cr} given by Equation F2-5) results in a maximum stress in the compression flange of $0.3F_y$.

In this Specification, the inelastic lateral-torsional buckling strength range for austenitic and duplex stainless steel I-shaped beams and channels is divided into two regions delimited by L_y, which is defined as the limiting unbraced length required to achieve the yield moment, as shown in Figure C-F1.2. This constitutes one of the most significant differences between the lateral-torsional buckling strength provisions given in this Specification and those given in the 2016 AISC *Specification* for carbon steel. L_y is determined by equating the yield moment, M_y, to the reduced elastic lateral-torsional buckling strength given by Equation F2-4 (with F_{cr} given by Equation F2-5), solving for L_b, and multiplying it by the reduction factor, $\beta_{y,LT}$, which accounts for the reduction in unbraced length due to material nonlinearity. The values of $\beta_{y,LT}$ were determined from numerical data on austenitic and duplex stainless steel I-shaped members provided by Fortan and Rossi (2021) and Marek and Jandera (2020).

For unbraced lengths ranging from L_p to L_y, the inelastic LTB strength of austenitic and duplex stainless steel compact I-shaped members and channels can be accurately approximated by a straight line that has as anchor points the plastic moment at L_p and the yield moment at L_y, as given by Equation F2-2. However, for unbraced lengths ranging from L_y to L_r, the inelastic LTB strength cannot be accurately approximated by a linear interpolation due to the significant nonlinearity exhibited by stainless steels prior to reaching the yield strength and the relatively large difference between L_r and L_y. Therefore, a slightly more complex expression is used that includes an exponential coefficient, α_{LT}, that varies with the length of the member relative to L_y and L_r, as given by Equations F2-3 and F2-6.

F3. DOUBLY SYMMETRIC I-SHAPED MEMBERS AND CHANNELS WITH COMPACT WEBS AND NONCOMPACT OR SLENDER FLANGES BENT ABOUT THEIR MAJOR AXIS

Section F3 is a supplement to Section F2 for the case where the flange of the section is either noncompact or slender. Figure C-F1.1 shows M_n varies linearly between λ_{pf} and λ_{rf}, in this range

$$0.33 \leq \frac{b_f/2t_f}{\sqrt{E/F_y}} \leq 0.41$$

addressing noncompact behavior and the curve beyond λ_{rf}, or greater than 0.41, addresses slender behavior.

Compared to the equivalent section in the 2016 AISC *Specification*, this section was extended to cover channels with compact webs and noncompact or slender flanges bent about the major axis. The reason for extending this section to channels is because several of the currently fabricated MC-shapes have flanges classified as nonslender or slender.

The main difference between the flexural strength provisions for the limit state of compression flange local buckling given in this section and those given in the corresponding section of the 2016 AISC *Specification* for carbon steel is due to the difference between the width-to-thickness limits of stainless steel and carbon steel that are used to define the flange as a compact, a noncompact, or a slender element, as shown in Figure C-F1.1. Another important difference, however, is the way in which the flexural buckling strength of doubly symmetric I-shaped members and channels with slender flanges is calculated. For carbon steel, the 2016 AISC *Specification* specifies that the flexural strength of I-shaped beams with slender flanges has to be determined based on the elastic local buckling stress of the compression flange, without consideration of post-buckling capacity. In this Specification, the flexural buckling strength of doubly symmetric I-shaped members and channels with slender flanges is calculated based on the effective section modulus determined with the effective width of the compression flange as given by Equation F3-3. This effective width expression was based on the research of Gardner and Theofanous (2008).

F4. DOUBLY SYMMETRIC I-SHAPED MEMBERS WITH NONCOMPACT WEBS BENT ABOUT THEIR MAJOR AXIS

The provisions of Section F4 are applicable to doubly symmetric I-shaped beams with noncompact webs. The flanges may be compact, noncompact, or slender.

Three limit states are considered in Section F4: (1) compression flange yielding; (2) lateral-torsional buckling; and (3) compression flange local buckling. The provisions for each of these limit states are based on the rules for doubly symmetric I-shaped beams given in Sections F2 and F3, but the flexural strength is limited by the web plastification factor, R_p, which accounts for the effect of inelastic local buckling of the web. R_p can vary from unity for doubly symmetric I-shaped beams with a web slenderness, $\lambda = \lambda_{rw}$, to as high as M_p/M_y for doubly symmetric I-shaped beams with a web slenderness, $\lambda = \lambda_{pw}$, for which the provisions are identical to those given in Sections F2 and F3.

F5. DOUBLY SYMMETRIC I-SHAPED MEMBERS WITH SLENDER WEBS BENT ABOUT THEIR MAJOR AXIS

This section applies to doubly symmetric I-shaped beams with a slender web. As is the case in Section F4, three limit states are considered: (a) compression flange yielding, (b) lateral-torsional buckling, and (c) compression flange local buckling.

This Specification adopted the same approach followed in the 2016 AISC *Specification* to account for the reduction in the flexural strength of carbon steel I-shaped beams with slender webs resulting from local buckling of the web. That is, local buckling of the web is considered through the bending strength reduction factor, R_{pg}. The expression used to calculate R_{pg} in this Specification was derived following the same methodology used by Basler and Thurlimann (1963) to derive the expression adopted in the 2016 AISC *Specification*. However, the expression was slightly rearranged so that the same expression can be used for austenitic and duplex stainless steel. The flexural strength data used as a benchmark for the derivation of R_{pg} was generated using the effective width expression proposed by Gardner and Theofanous (2008) for stiffened elements with the elastic local buckling stress calculated using a plate buckling coefficient, k = 23.9. The reduction in flexural strength obtained using the reduction factor, R_{pg}, and the effective width method differ by less than 2% for web slenderness, $\lambda \leq 1.5\lambda_{rw}$. For larger web slendernesses, the use of R_{pg} becomes very conservative.

Alternatively, I-shaped members with a cross-section slenderness, $\lambda_l \leq 1.6$, and an unbraced length, $L_b \leq 0.5L_p$, where L_p is given by Equation F2-7, can be designed in accordance with the CSM included in Appendix 2; otherwise, the users may also want to consider the applicability of the *Specification for the Design of Cold-Formed Stainless Steel Structural Members* (ASCE, 2021), which provides additional methods for locally slender cross sections made of cold-formed stainless steel.

As opposed to the approach followed in the 2016 AISC *Specification*, in this Specification the critical stress, F_{cr}, for doubly symmetric I-shaped beams with a slender web is calculated using the same expression (Equation F2-5) used to calculate F_{cr} for doubly symmetric I-shaped beams with a compact or noncompact web. This is to avoid a discontinuity in the lateral-torsional buckling strength, such as the one that results from the provisions in the 2016 AISC *Specification*, as the web transitions from a noncompact to a slender element.

F6. I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MINOR AXIS

I-shaped members and channels bent about their minor axis do not experience lateral-torsional buckling. For I-shaped members, the only limit states to consider are yielding and flange local buckling. For channels, the limit states to consider depend on the direction of the minor-axis bending. If the direction of bending is such that it results in compressive stresses in the tip of the channel flanges, the limit states to consider are yielding and flange local buckling. If, on the other hand, bending results in compressive stresses in the web of the channel, the limit states to consider are yielding and web local buckling. This distinction represents one of the main differences with respect to the provisions given in the equivalent section of the 2016 AISC *Specification*, where the limit states considered are yielding and flange local buckling irrespective of the direction of bending, and the limit state of web local buckling is ignored.

Another difference with respect to the provisions for I-shaped members and channels bent about the minor axis in the 2016 AISC *Specification* is that for the limit state of yielding, the upper bound limit of $1.6F_yS_y$ specified in the 2016 AISC *Specification* on the plastic moment was not incorporated into this Specification because when these types of members are made of stainless steel, the significant strain hardening exhibited by the material allows for the full plastic moment to be attained before the ultimate strain is reached at the extreme fibers of the cross section.

The limiting width-to-thickness ratios for the flanges of I-shaped members and channels given in Table B4.1b are the same for major- and minor-axis bending. This is a simplification similar to that adopted in the 2016 AISC *Specificatio*n that leads to conservative strength predictions for the limit state of flange local buckling. However, for channels bent about the minor axis, the local buckling strength provisions in this Specification are less conservative than those in the 2016 AISC *Specification* because they only apply to channels bent with the flange tips in compression.

For channels with noncompact webs and bent with the web in compression, web local buckling is accounted for by assuming a linear reduction of the flexural strength from the plastic moment when the web slenderness is equal to λ_{pw}, to the yield moment when the web slenderness is equal to λ_{rw}. This is a conservative simplification, which, although it does not reflect the mechanical behavior of the cross section, provides a simple means to design against web local buckling. In fact, channels with a web slenderness equal to λ_{rw} are able to develop a flexural strength larger than the yield moment before failing due to local buckling of the web, because when the channel reaches its yield moment strength, yielding takes place on the tension side at the flange tip, while the stresses in the web are still below the specified minimum yield stress. This means that channels with slender webs may still be able to reach the yield moment, as reflected by Equation F6-5 which is applicable for channels with slender webs that don't exceed $1.5\lambda_{rw}$.

F7. SQUARE AND RECTANGULAR HSS AND BOX SECTIONS

This section applies to square and rectangular HSS and doubly symmetric box sections with $h/b \leq 3$ bent about either axis, having compact, noncompact, or slender webs and flanges, as defined in Section B4.1 for flexure. The provisions for the nominal flexural strength of these types of cross sections include the limit states of yielding, flange local buckling, and web local buckling. No provisions are given for design against lateral-torsional buckling because in most practical cases, rectangular HSS and doubly symmetric box sections with $h/b \leq 3$ will not be susceptible to this limit state.

The provisions for local buckling of noncompact HSS and box sections follow the same format as those in the 2016 AISC *Specification*, with the only difference being the limiting width-to-thickness parameters of the flange and web as given in Table B4.1b.

For rectangular HSS and box sections with slender flanges, the nominal flexural strength for the limit state of flange local buckling is determined from the effective section modulus calculated using the effective width of the compression flange when the stress in the corners is at yield. This approach is the same as the one used in the 2016 AISC *Specification*. However, the effective width expression used in this Specification differs from the one used in the 2016 AISC *Specification*, and it is based on the research of Gardner and Theofanous (2008).

The effective width expression given by Equation F3-3, used for the determination of S_e used in Equation F7-3, does not include the increase in strength at the corners of rectangular HSS resulting from cold forming. Therefore, the same equation is used to calculate the compression flange effective width of welded box sections and cold-formed rectangular HSS. For cold-formed rectangular HSS, Equation F3-3 can be used in conjunction with the average yield strength, $F_{y,avg}$, given in Section B4.3 for these types of members.

Alternatively, the effective width of cold-formed rectangular HSS can be determined using Equation F7-4. However, if this equation is used, the effective width and the flexural strength have to be calculated using the specified minimum yield stress, F_y.

Web local buckling of rectangular HSS and box sections with slender webs is accounted for using the same approach adopted in the 2016 AISC *Specification*, where the effective or elastic section modulus of the cross section, depending on whether the compression flange is slender or not, is reduced by the bending strength reduction factor, R_{pg}. This reduction factor was developed for austenitic and duplex stainless steel I-sections with slender webs, as explained in Section F5 of this Commentary, and therefore, for rectangular HSS and box sections, it is calculated with a doubling of a_w to account for two webs.

F8. ROUND HSS

The flexural strength provisions for austenitic and duplex stainless steel round HSS are limited to compact and noncompact cross sections.

The expression included in the 2016 AISC *Specification* for local buckling of noncompact round HSS is:

$$M_n = \left[\frac{0.021E}{\left(\dfrac{D}{t}\right)} + F_y \right] S \tag{C-F8-1}$$

As the λ_p and λ_r values for round HSS in bending are practically identical for carbon steel and stainless steel, Equation C-F8-1 is adopted for stainless steel but presented in a slightly different format.

Although Chapter F does not cover slender round HSS in flexure, these members can be designed in accordance with the CSM included in Appendix 2 when $D/t \leq 0.44\,E/F_y$ (or $\lambda_l \leq 0.60$).

F9. SOLID RECTANGULAR SHAPES AND ROUNDS

The provisions in Section F9 apply to solid shapes with a round or rectangular cross section. The prevalent limit state for such members is the attainment of the full plastic moment, M_p. The upper bound limit of $1.6F_yS_y$ for the plastic moment of solid rectangular shapes and rounds made of carbon steel in the 2016 AISC *Specification* was removed from this Specification because solid rectangular shapes and rounds made of stainless steel are able to develop their full plastic moment capacity before the extreme fibers reach the ultimate strain due to the significant strain hardening exhibited by the material.

Solid rectangular shape members where the depth is larger than the width may also be susceptible to lateral-torsional buckling. The format of the lateral-torsional buckling strength curve adopted in this Specification is the same as the one used in the 2016 AISC *Specification* for solid rectangular shape members made of carbon steel. However, the curve was slightly reduced, while the limiting length for which the limit state of lateral-torsional buckling is not applicable was increased. The adjustments to the lateral-torsional buckling curve were based on the results from numerical simulations on solid rectangular shape members made of austenitic and duplex stainless steel, and they reflect the effect of larger material nonlinearity and strain hardening in stainless steel compared to carbon steel.

F10. OTHER SHAPES

The provisions in this section apply to all shapes without slender elements that are not covered in the other sections of Chapter F with the exception of single angles.

The provisions in this section are more restrictive than those encountered in the other sections of Chapter F. The flexural strength is limited to the yield moment, as opposed to the plastic moment. The lateral-torsional buckling strength is determined using the same curve given in Section F2 for doubly symmetric I-shaped members and channels made of austenitic alloys, which is lower than that for duplex alloys. The only difference is that no expression is given for determining the critical stress, F_{cr}, due to the impossibility of developing a single and simple expression that is applicable to a member with arbitrary shape. Instead, in order to determine the flexural strength for the limit state of lateral-torsional buckling, the design engineer needs to resort to principles of structural mechanics, textbooks, or handbooks, such as the SSRC Guide (Ziemian, 2010), papers in journals, or finite element analyses to determine F_{cr} and then back-calculate L_y and L_r.

F11. PROPORTIONS OF BEAMS AND GIRDERS

Compared to the 2016 AISC *Specification*, this Specification does not give proportioning limits for I-shaped members. The proportioning limits given in the 2016 AISC *Specification* are not based on strength, and the provisions given in this Specification provide reliable strength predictions without adopting these limits. Therefore, given the higher cost of stainless steel and its unique applications it was deemed unnecessary to restrict its proportions to those commonly used in carbon steel building construction.

In addition, this Specification does not give provisions for cover plates or unbraced length for moment redistribution.

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

The provisions for proportions of rolled beams and girders with holes in the tension flange are almost identical to those included in the 2016 AISC *Specification*, and they are based on research from Dexter and Altstadt (2004) and Yuan et al. (2004) that indicates that the flexural strength of the net section can be predicted by comparison of the quantities F_yA_{fg} and F_uA_{fn}. The only difference between the stainless steel and carbon steel provisions is that the parameter Y_t, which is used in the 2016 AISC *Specification* to account for the difference between F_y and F_u, was set to 1.0 in this Specification because the ratio of F_y to F_u is less than 0.8 for any type of stainless steel.

The resistance factor and safety factor used throughout this chapter, $\phi = 0.90$ and $\Omega = 1.67$, are those normally applied for the limit state of yielding. In the case of rupture of the tension flange due to the presence of holes, the provisions of this chapter continue to apply the same resistance and safety factors.

CHAPTER G

DESIGN OF MEMBERS FOR SHEAR AND TORSION

Compared to the 2016 AISC *Specification*, this Specification does not include provisions for determining the shear strength of single angles due to the lack of research and test data for these types of members. In addition to that, the provisions for members subject to torsion only, which in the 2016 AISC *Specification* are included in the same chapter dealing with members subject to combined loading, were moved to this chapter as it was considered that torsion is more akin to shear.

G1. GENERAL PROVISIONS

The resistance factors used with the provisions of this chapter were obtained from a reliability analysis carried out by Chen et al. (2020a) based on test data reported by Unosson and Olsson (2003), Real et al. (2007), Estrada et al. (2007), Chen et al. (2018), Estrada et al. (2008), and Saliba and Gardner (2013) on austenitic and duplex stainless steel welded I-shaped members subject to shear buckling of the web. The resistance factors obtained for austenitic and duplex stainless steel I-shaped members were larger than 1.019 when tension field action is not taken into account in the shear buckling resistance, while they were larger than 0.962 when tension field action is considered. However, in order to maintain consistency with the 2016 AISC *Specification*, a resistance factor of 0.90 was adopted for both cases.

Because there are no experimental data available for the members covered in Section G3 and Section G5, a resistance factor of 0.90 was adopted in combination with a more conservative shear buckling curve composed of three segments.

G2. I-SHAPED MEMBERS AND CHANNELS SUBJECT TO MAJOR-AXIS SHEAR

1. Shear Strength of Webs without Tension Field Action

Section G2.1 applies to I-shaped members and channels with unstiffened webs, I-shaped members and channels with transverse stiffeners spaced wider than $3h$, and end panels of I-shaped members and channels with transverse stiffeners spaced closer than $3h$. The provisions in this section apply when post-buckling strength develops due to web stress redistribution but classical tension field action is not developed. They may be conservatively applied where it is desired to not use the tension field action enhancement for convenience in design.

The nominal shear strength of a web is defined by Equation G2-1 as a product of the shear yield force, $0.6F_yA_w$, and the shear post-buckling strength reduction factor, C_{v1} (referred to as the web shear strength coefficient in the Specification). This Specification follows the same philosophy adopted in the 2016 AISC *Specification* to calculate the shear strength of a web when tension field action is not considered.

However, the equations for determining the shear post-buckling strength reduction factor, C_{v1}, given in the 2016 AISC *Specification* were modified based on the research from Chen et al. (2020a) to account for the larger material nonlinearity and strain hardening exhibited by stainless steel in comparison to carbon steel.

Webs of austenitic and duplex stainless steel I-shaped members and channels with low slenderness are able to reach a shear strength that is larger than the yield stress, $0.6F_y$, due to the development of strain hardening. This is accounted for in this Specification by allowing the C_{v1} coefficient to reach values larger than 1.0, for $\lambda \leq 0.85\sqrt{k_v E/F_y}$, as given by Equations G2-2 and G2-3. For intermediate slenderness, the provisions in the 2016 AISC *Specification* for calculating the shear strength of web panels without consideration of tension field action overestimate the shear strength of stainless steel web panels due to the pronounced material nonlinearity exhibited by stainless steel. For webs with high slenderness, on the other hand, the provisions in the 2016 AISC *Specification* are relatively conservative when compared against experimental data on stainless steel I-shaped members failing by shear buckling of the web (Chen et al., 2020a). To account for this, Equation G2-3 in this Specification predicts shear strengths lower than those given by the provisions in the 2016 AISC *Specification* for web slenderness, λ, ranging between $0.85\sqrt{k_v E/F_y}$ and $1.71\sqrt{k_v E/F_y}$, while for higher slenderness the shear strengths predicted by Equation G2-3 are slightly larger.

The provisions for calculating the web plate shear buckling coefficient, k_v, for panels subject to pure shear, with and without transverse stiffeners, are the same as those included in the 2016 AISC *Specification*.

2. Shear Strength of Interior Web Panels with $a/h \leq 3$ Considering Tension Field Action

Section G2.2 applies to interior panels of I-shaped members and channels with stiffeners spaced at $3h$ or smaller.

This Specification adopted the same approach used in the 2016 AISC *Specification* to account for tension field action. Basler's equation, which is used to account for full tension field action in doubly and singly symmetric I-shaped members and channels with $2A_w/\left(A_{fc}+A_{ft}\right) \leq 2.5$, $h/b_{fc} \leq 6.0$, and $h/b_{ft} \leq 6.0$ is given by Equation G2-6, while for beams that do not meet these geometrical requirements, Equation G2-7 gives an expression based on partial tension field action. The only difference between the provisions in this Specification and those given in the 2016 AISC *Specification* for calculating the shear strength of web panels considering tension field action is that the web shear buckling coefficient, C_{v2}, was modified to take into account the strain hardening and material nonlinearity of stainless steel alloys based on the expressions given by Real et al. (2007) and later modified by Chen et al. (2020a).

The expressions for C_{v2} are divided into four zones (Figure C-G2.1), according to different buckling stress levels. Zone 1 corresponds to the case in which the yield strength is attained in the web panel before the occurrence of shear buckling, and

$C_{v2} > 1.0$ due to the beneficial effect of strain hardening. In Zone 2, the critical shear buckling stress is heavily influenced by the material nonlinearity, which leads to a noticeable strength reduction. In this zone, the expression for calculating C_{v2} was derived iteratively using the plasticity reduction factor, $\eta = \sqrt{G_t/G}$, where G_t is the tangent shear modulus and G is the initial shear modulus. In Zone 3, the shear buckling stress is somewhat influenced by the material nonlinearity, but its effect is not as marked as in Zone 2. The expression for C_{v2} in this zone was derived by fitting the points in the upper and lower limits while keeping the same slope at the lower limit with the expression for C_{v2} in Zone 4. In Zone 4, the shear buckling stress is rather low and is not influenced by the material nonlinearity, and the expression for C_{v2} coincides with the critical elastic shear buckling stress.

3. Transverse Stiffeners

Transverse stiffeners in I-girders designed for shear post-buckling strength, including tension field action, are loaded predominantly in bending due to the restraint they provide to lateral deflection of the web.

The requirements on the flexural rigidity the transverse stiffeners must provide in order to prevent lateral deflection of the web due to shear buckling are the same as those prescribed in the 2016 AISC *Specification*. However, the limiting slenderness requirement to avoid local buckling of the stiffener was reduced to reflect the detrimental effect of the increased nonlinearity of stainless steel compared to carbon steel.

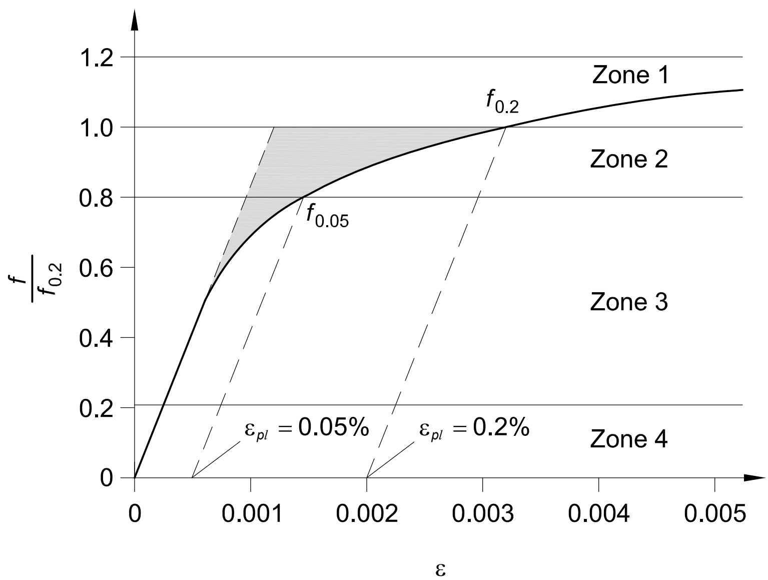


Fig. C-G2.1. Stress zones.

G3. RECTANGULAR HSS AND BOX SECTIONS SUBJECT TO SHEAR

The provisions in this Specification for calculating the shear strength of austenitic and duplex stainless steel rectangular HSS and box sections follow the same approach used in the 2016 AISC *Specification*, where the shear strength is calculated using the web shear buckling coefficient, C_{v2}, given in Section G2.2, and the web plate shear buckling coefficient, $k_v = 5$. These provisions can be expected to result in conservative estimations of the shear resistance of the webs of HSS and box sections as they do not account for shear post-buckling strength.

G4. ROUND HSS SUBJECT TO SHEAR

The provisions for austenitic and duplex stainless steel round HSS subject to shear were derived based on the design methodology followed by the 2016 AISC *Specification* and taking into account the recommendation of Gerard (1957) to account for the gradually yielding stress-strain response of stainless steel alloys.

Little information is available on round HSS subjected to transverse shear. Therefore, the provisions in the 2016 AISC *Specification* are based on local buckling of cylinders due to torsion. However, since torsion is generally constant along the member length and transverse shear usually has a gradient, the critical stress for transverse shear is taken as 1.3 times the critical stress for torsion (Brockenbrough and Johnston, 1981; Ziemian, 2010). The torsion equations apply over the full length of the member, but for transverse shear it is reasonable to use the length between the points of maximum and zero shear force.

The 2016 AISC *Specification* stipulates that the shear buckling strength of a round HSS subject to shear should be taken as the larger of the elastic shear buckling stress computed for a round HSS with intermediate length [$100 \le Z \le 19.2\left(1-\nu^2\right)\left(D/t\right)^2$] and long length [$Z > 19.2\left(1-\nu^2\right)\left(D/t\right)^2$]. The parameter Z, which is used to distinguish between round HSS of medium and long lengths, is given by:

$$Z = 2\left(\frac{L_v}{D}\right)^2\left(\frac{D}{t}\right)\sqrt{1-\nu^2} \tag{C-G4-1}$$

where ν is Poisson's ratio, D is the diameter of the cross section, t is the thickness, and L_v is the portion of the member length subject to shear. For round HSS with medium and long lengths, the critical elastic shear buckling stresses, F_{cr}, included in the 2016 AISC *Specification* are given by Equations C-G4-2 and C-G4-3, respectively:

$$F_{cr} = \frac{1.60E}{\sqrt{\frac{L_v}{D}}\left(\frac{D}{t}\right)^{5/4}} \tag{C-G4-2}$$

$$F_{cr} = \frac{0.78E}{\left(\frac{D}{t}\right)^{3/2}} \tag{C-G4-3}$$

For materials that experience gradual yielding, such as stainless steel, Gerard (1957) showed that the shear buckling stress in the inelastic range can be obtained by replacing the modulus of elasticity, E, in Equations C-G4-2 and C-G4-3 by the secant modulus at a stress, f, equal to $2F_{cr.}$

In order to avoid the iterations associated with the use of the secant modulus, the equations for determining the shear buckling strength coefficients for austenitic and duplex stainless steel round HSS of medium length, C_{vM}, and long lengths, C_{vL}, are based on the initial modulus of elasticity. The expressions for determining these coefficients were developed by generating shear strength predictions using the iterative approach suggested by Gerard (1957) and using these predictions as benchmarks for fitting these expressions. Data points were generated for round HSS of medium and long lengths and covering cross sections with a wide range of D/t ratios. The generated data included round HSS made of austenitic stainless steel alloys S30400/S30403 and S31600/S31603 and duplex stainless steel alloys S32101, S32202, S32205, and S32304, which constitute the most commonly used stainless steel alloys for structural applications.

By defining the shear buckling slenderness, λ_s, of round HSS of medium and long length as:

$$\lambda_s = \sqrt{\frac{0.6F_y}{F_{s,el}}} \tag{C-G4-4}$$

where $F_{s,el}$ is the elastic shear buckling stress given by Equation C-G4-2 for round HSS of medium lengths and Equation C-G4-3 for round HSS of long lengths, the shear strength coefficient for stainless steel round HSS of medium and long lengths can be written as follows.

When $\lambda_s \le 0.30$

$$C_{vM} = C_{vL} = 1.0 \tag{C-G4-5}$$

When $0.30 < \lambda_s \le 1.60$

$$C_{vM} = C_{vL} = 1.14 - 0.47\lambda_s \tag{C-G4-6}$$

When $\lambda_s > 1.60$

$$C_{vM} = C_{vL} = \frac{1.0}{\lambda_s} \tag{C-G4-7}$$

Equations C-G4-2 to C-G4-7 are an alternative way of presenting the provisions in Section G4 given by Equations G4-4 to G4-9.

Figure C-G4.1 shows the relationship between the shear strength coefficients for austenitic and duplex round HSS of medium and long lengths and the shear buckling slenderness, λ_s, and compares them against the corresponding shear strength coefficient for carbon steel. For high shear slenderness ($\lambda_s > 1.60$), the shear buckling response of austenitic and duplex stainless steel round HSS is considered to be elastic, and the shear buckling strength given in this Specification coincides with the one given in the 2016 AISC *Specification* for carbon steel. However, for intermediate slenderness ($0.30 < \lambda_s \le 1.60$), due to the loss of stiffness exhibited by stainless steel as the critical shear stress approaches the yield stress, the provisions in this

Specification predict shear buckling strengths that are significantly lower than those for carbon steel. When the shear buckling slenderness of austenitic and duplex round HSS is less than 0.3, the shear strength is considered to be governed by shear yielding, and the shear strength coefficients given in this Specification are equal to 1.0.

Given that the shear buckling strength of austenitic and duplex stainless steel round HSS of medium and long lengths can be determined using the same strength curve (given by Equations C-G4-4 to C-G4-7), by comparing the shear buckling slenderness for round HSS of medium and long lengths, it can be found that when $L_v/D \leq 4.21\sqrt{D/t}$, the shear strength is governed by C_{vM}; otherwise, the shear strength is governed by C_{vL}.

In the equation for the nominal shear strength, V_n, it is assumed that the shear stress at the neutral axis, VQ/Ib, is at $0.6F_yC_v$. For a thin round section with radius R and thickness t, $I = \pi D^3 t$, $Q = 2R^2 t$, and $b = 2t$. This gives the stress at the centroid as $V/\pi Rt$, in which the denominator is recognized as half the area of the round HSS.

G5. DOUBLY SYMMETRIC I-SHAPED MEMBERS AND CHANNELS SUBJECT TO MINOR-AXIS SHEAR

The provisions of this Specification for calculating the weak-axis shear strength of I-shaped member and channel flanges follow the same approach used in the 2016 AISC *Specification*, where the shear strength is conservatively calculated using the web shear buckling coefficient, C_{v2}, given in Section G2.2, which does not account for shear post-buckling strength, and the plate buckling coefficient, $k_v = 1.2$, to account for the presence of a free edge.

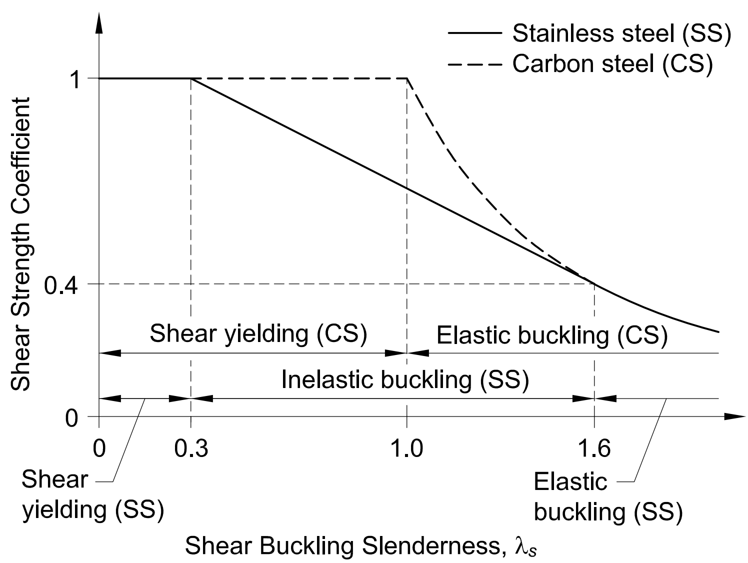


Fig. C-G4.1. Shear buckling coefficients for austenitic and duplex stainless steel, and carbon steel round HSS of medium and long lengths subject to transverse shear.

G6. OTHER SINGLY OR DOUBLY SYMMETRIC SHAPES SUBJECT TO SHEAR

This Specification follows the same approach used in the 2016 AISC *Specification* to calculate the shear strength of singly or doubly symmetric shapes that are not covered by the other sections of Chapter G.

The shear strength of the section is determined by adding the contribution from each element resisting the shear force. Post-buckling strength from Section G2.1 is not included due to lack of experimental verification.

G7. BEAMS AND GIRDERS WITH WEB OPENINGS SUBJECT TO SHEAR

This Specification adopted the same requirements included in the 2016 AISC *Specification* to account for the effect of web openings on the shear strength of beams and girders.

G8. ROUND AND RECTANGULAR HSS AND BOX SECTIONS SUBJECT TO TORSION

The provisions for austenitic and duplex stainless steel HSS and box sections subject to torsion were derived based on the torsion strength provisions given in the 2016 AISC *Specification* for carbon steel HSS and box sections, and following the same approach used to derive the shear strength provisions for austenitic and duplex stainless steel round HSS given in Section G4 and rectangular HSS and box sections given in Section G3.

The pure torsional shear stress in HSS and box sections is assumed to be uniformly distributed along the wall of the cross section, and it is equal to the torsional moment divided by a torsional constant for the cross section, C. In a limit state format, the nominal torsional resisting moment is the shear constant times the critical shear stress, which can be written as $0.6C_vF_y$.

For round HSS, the torsional constant is equal to the polar moment of inertia divided by the radius, as given in the equation for C in the User Note of Section G8.

For rectangular HSS, the torsional constant is obtained as $2tA_o$ using the membrane analogy (Timoshenko, 1956), where A_o is the area bounded by the midline of the section. An outside corner radius of 2_t and a midline radius of $1.5t$ are assumed, and

$$A_o = \left(B - t\right)\left(H - t\right) - 9t^2\frac{\left(4-\pi\right)}{4} \tag{C-G8-1}$$

resulting in the equation for C in the User Note of Section G8.

1. Round HSS

The shear buckling strength coefficients, C_{vM} and C_{vL}, for austenitic and duplex stainless steel round HSS subject to torsion were derived following the same approach used to derive the shear buckling coefficients for round HSS subject to transverse shear. That is, they are based on torsional strength data generated by computing the critical elastic buckling strength using the equations given in the 2016 AISC *Specification* for round HSS of medium (Equation C-G8-2) and long (Equation C-G8-3) lengths, but replacing the initial modulus of elasticity with the secant modulus calculated at a stress, f, equal to $2F_{cr}$ in order to account for the gradual yielding of stainless steels (Gerard, 1957). Equation C-G8-2 is given in Schilling (1965) and includes a 15% reduction to account for initial imperfections. Equation C-G8-3 is based on the expression by Timoshenko and Gere (1961) but is reduced by 15%, as recommended in the 2016 AISC *Specification*, to account for initial imperfections.

$$F_{cr} = \frac{1.23E}{\sqrt{\frac{L}{D}}\left(\frac{D}{t}\right)^{5/4}} \tag{C-G8-2}$$

$$F_{cr} = \frac{0.6E}{\left(\frac{D}{t}\right)^{3/2}} \tag{C-G8-3}$$

2. Rectangular HSS and Box Sections

The provisions for determining the shear buckling strength coefficient, C_v, for rectangular HSS and box sections subject to torsion are identical to those given in Section G2.2 to calculate C_{v2} with the shear buckling coefficient equal to $k_v = 5.0$. The shear distribution due to torsion is uniform in the longest sides of a rectangular HSS, and this is the same distribution that is assumed to exist in the web of an I-shape beam. Therefore, it is reasonable that the provisions for buckling are the same in both cases.

G9. DOUBLY SYMMETRIC I-SHAPED MEMBERS AND CHANNELS SUBJECT TO TORSION

This Specification adopted the same provisions given in the 2016 AISC *Specification* to calculate the torsion strength for doubly symmetric I-shaped members and channels.

CHAPTER H

DESIGN OF MEMBERS FOR COMBINED FORCES

Chapters D, E, F, and G of this Specification address austenitic and duplex stainless steel members subject to only one type of force: axial tension, axial compression, flexure, and shear and torsion, respectively, or to multiple forces that can be treated as only one type of force. This chapter addresses members subject to a combination of two or more individual forces.

The provisions of this chapter for the design of members subject to combined loading are the same as those included in the 2016 AISC *Specification* for doubly and singly symmetric members subject to combined loading. The provisions for members subject to combined flexure and axial force were compared against test results on round HSS and I-shaped members reported in the SCI *Commentary to the Design Manual for Structural Stainless Steel* (Baddoo, 2018) and test results on rectangular HSS reported in Young and Hartono (2002). The test results lie above the interaction curve in all cases. A resistance factor larger than 0.90 was calculated for austenitic stainless steel. However, in order to maintain consistency with the 2016 AISC *Specification*, a resistance factor of 0.90 was adopted. No data were available for duplex stainless steel; therefore, a resistance factor of 0.90 is assumed, based upon the results for austenitic stainless steel.

Recent investigations into the interaction between induced shear stress and flexural stress within stainless steel plate girders under lateral load indicate this interaction is sufficiently insignificant as to be safely neglected for most practical design considerations (Chen et al., 2020b). While some amount of interaction is present under low stress scenarios, plastic mechanisms occurring under ultimate limit state conditions have been found to be largely insensitive to this interaction.

The continuous strength method (CSM), included in Appendix 2, provides an alternative, less conservative method for determining the strength of laterally restrained I-shaped, HSS, and box-section members subject to compression and flexure about one or both axes, when the requirements given in Appendix 2, Table A-2.1.1, are met.

H1. DOUBLY SYMMETRIC I-SHAPED MEMBERS, CHANNELS, HSS, AND BOX SECTIONS SUBJECT TO FLEXURE AND AXIAL FORCE

1. Doubly Symmetric I-Shaped Members, Channels, HSS, and Box Sections Subject to Flexure and Compression

This section contains design provisions for doubly symmetric I-shaped members, channels, and HSS under combined flexure and compression and under combined flexure and tension.

The normalized equations corresponding to a beam-column are as follows:

$$\frac{P_r}{P_c}+\frac{8}{9}\frac{M_r}{M_c}=1 \quad \text{for } \frac{P_r}{P_c}\geq 0.2 \tag{C-H1-1}$$

$$\frac{P_r}{2P_c}+\frac{M_r}{M_c}=1 \quad \text{for } \frac{P_r}{P_c}< 0.2 \tag{C-H1-2}$$

The interaction equations in this section (Equations H1-1a and H1-1b) are designed to be very versatile. The terms in the denominator fix the endpoints of the interaction curve. The available flexural strength, M_c, is determined by the appropriate provisions from Chapter F. It encompasses the limit states of yielding, lateral-torsional buckling, flange local buckling, and web local buckling.

The axial term, P_c, is governed by the provisions of Chapter E, and it can accommodate nonslender or slender-element columns, as well as the limit states of major- and minor-axis flexural buckling, and torsional and flexural-torsional buckling.

The utility of the interaction equations is further enhanced by the fact that they also permit the consideration of biaxial bending without the presence of axial load.

2. Doubly Symmetric I-Shaped Members, Channels, HSS, and Box Sections Subject to Flexure and Tension

Section H1.1 considers the most frequently occurring cases in design: members under flexure and axial compression. Section H1.2 addresses the less frequent cases of flexure and axial tension. Since axial tension increases the bending stiffness of the member to some extent, Section H1.2 permits the increase of C_b in Chapter F. Thus, when the bending term is controlled by lateral-torsional buckling, the moment gradient factor, C_b, is increased by:

$$\sqrt{1+\frac{\alpha P_r}{P_{ey}}}$$

H2. DOUBLY SYMMETRIC I-SHAPED MEMBERS, CHANNELS, HSS, AND BOX SECTIONS SUBJECT TO COMBINED TORSION, FLEXURE, SHEAR, AND/OR AXIAL FORCE

1. HSS and Box Sections Subject to Combined Torsion, Shear, Flexure, and Axial Force

The interaction equation for austenitic and duplex HSS is the same as the one used in the 2016 AISC *Specification*. In this interaction equation, normal effects due to flexural and axial load effects are combined linearly and then combined with the square of the linear combination of flexural and torsional shear effects. When an axial compressive load effect is present, the required flexural strength is to be determined by second-order analysis. When normal effects due to flexural and axial load effects are not present, the square of the linear combination of flexural and torsional shear effects underestimates the actual interaction. A more accurate measure is obtained without squaring this combination.

2. Doubly Symmetric I-Shaped Members and Channels Subject to Combined Stress

The required stresses are determined by elastic stress analysis based on established theories of structural mechanics. The three limit states to consider and the corresponding available stresses are:

(a) Yielding under normal stress—F_y

(b) Yielding under shear stress—$0.6F_y$

(c) Buckling—F_{cr}

In most cases, it is sufficient to consider normal stresses and shear stresses separately because maximum values rarely occur in the same place in the cross section or at the same place in the span.

The provisions in this section allow constrained local yielding to take place adjacent to areas that remain elastic without constituting failure of the member.

H3. RUPTURE OF FLANGES WITH HOLES SUBJECT TO TENSION

Equation H3-1 is provided to evaluate the limit state of tensile rupture of the flanges of beam-columns. This provision is only applicable in cases where there are one or more bolt holes in the flange in net tension under the combined effect of flexure and axial forces. When both the axial and flexural stresses are tensile, their effects are additive. When the stresses are of opposite sign, the tensile effect is reduced by the compression effect.

CHAPTER I

DESIGN OF COMPOSITE MEMBERS

Steel-concrete composite construction is very popular owing to the efficient use of the two constituent materials. There are examples of structures made from concrete-filled hollow stainless steel sections and also stainless steel composite bridges, where the stainless steel beam is connected to the concrete slab through steel headed stud anchors. However, to date, no international design standard gives design rules for stainless steel-concrete composite design, although this is likely to change in the next 10 years.

There has been a fair amount of research into the behavior of stainless steel-concrete composite columns over the last 15 years (e.g., Ellobody and Young, 2006; Lam and Gardner, 2008; Uy et al., 2011; Tao et al., 2011; Liao et al., 2017; and Han et al., 2019). The behavior of stainless steel-concrete composite beams has been studied numerically and analytically (Shamass and Cashell, 2019), although there are no tests currently available in the literature. Composite joints have also been studied (Song at al., 2019).

In principle, the same methods given in Chapter I of the 2016 AISC *Specification* for carbon steel composite construction will apply to stainless steel, although some adjustments to coefficients will be required to provide strain compatibility arising from the nonlinear stress-strain characteristics of stainless steel.

The issue of shear connection is paramount for all composite construction, and it is recommended to use welded stainless steel headed stud anchors to maintain the corrosion resistance throughout the entire system. The studs should be made from cold-drawn bar stock conforming to ASTM A493 (ASTM, 2016c) or ASTM A276/A276M (ASTM, 2017a). AWS D1.6/D1.6M (AWS, 2017) covers stud welding:

(1) Stainless steel studs to stainless steel base metal

(2) Stainless steel studs to carbon steel base metal

(3) Carbon steel studs to stainless steel base metal

Consideration should be given to the impact of any loss of stiffness of the headed stud anchor (resulting from the nonlinear stress-strain characteristics of stainless steel) on the beam or column and connection behavior. The loss of stiffness will be greater for composite slabs with trapezoidal decking compared to plain concrete slabs (typically used for bridges).

CHAPTER J
DESIGN OF CONNECTIONS

The provisions of Chapter J cover the design of connections not subject to cyclic loads. Wind and other environmental loads are generally not considered to be cyclic loads. The provisions generally apply to connections other than HSS and box sections. See Chapter K for provisions specific to HSS and box-section connections and Appendix 3 for fatigue provisions.

J1. GENERAL PROVISIONS

The provisions in this section were adopted without modification from the 2016 AISC *Specification*, except as detailed in this Commentary.

5. Splices in Heavy Sections

For the reasons listed in Commentary Section J2, careful joint fit-up, preparation, joint location, and weld sequence are critical for minimizing distortion. Strong fixtures and tooling are needed to prevent movement during welding, which is a particular concern with austenitic stainless steels. Many small beads will cause greater distortion than a few heavy beads.

When inert gas shielding is used, the tooling should also provide an inert gas backup at the root of the weld to prevent oxidation when the root pass is being made. While the shielding gas is usually argon, helium or mixtures of argon and helium are used for heavy sections.

Due to the differences between carbon and stainless steels, the volume of weld metal in joints must be limited to the smallest size that will provide the necessary properties. In thick plate, a “U” groove gives a smaller weld volume than a “V” groove and should be considered. If it is possible to weld from both sides of a joint, a double “U” or “V” groove joint preparation should be used. This not only reduces the volume of weld metal required but also helps to balance the shrinkage stresses.

If copper chills (a type of heat sink to accelerate cooling near welds) are to be used near a weld area, they should be nickel plated to prevent copper pickup. If copper is in contact with the high-temperature region of the heat affected zone, it can melt and penetrate the grain boundaries of austenitic stainless steel, causing cracking.

AWS D1.6/1.6M, clause 7 (AWS, 2017), requires that all shop welded splices in each component be made prior to welding component parts to other component parts of the member. When making subassembly parts in the shop or field, the welding sequence is required to be reasonably balanced between the web and flange welds as well as about the major and minor axes of the member. The drawings are required to show the procedures and welding sequences to minimize distortion and shrinkage stresses. AWS D1.6/D1.6M, clause 4, Part C, defines the requirements, limitations, and prohibitions for splices.

The interior portions of heavy plates may contain a coarser grain structure and/or lower notch toughness than other areas of these products. The toughness of stainless steel is superior to that of carbon steel, particularly for the austenitic stainless steels.

The provisions of AWS D1.6/D1.6M are minimum requirements that apply to most structural welding situations. Guidance specific to the higher alloyed austenitic stainless steels and any of the duplex stainless steels should be obtained from the stainless steel mill. For some alloys, there will be plate thickness restrictions for heavy welded sections; specifically, the maximum duplex plate thickness that can be welded is typically 2 in. (50 mm).

Note that while AWS D1.6/D1.6M provides prequalified weld procedure specifications (PWPS) for standard austenitic stainless steels, these do not exist for duplex or high alloy stainless steels, and the fabricator must create their own welding procedure specification (WPS) for various thickness ranges when using these materials. It is also necessary to establish a procedure qualification record (PQR) for various thickness ranges when using these materials. Additional welding time and budget (when compared to austenitic stainless steels) is required when using these materials.

6. Beam Copes and Weld Access Holes

The provisions given in the Specification are taken from AWS D1.6/AWS D1.6M, clauses 7.15, 7.4.7.1, 7.4.7.2, and Figure 7.1.

Where practical, reentrant corner or cut materials should be formed to provide a gradual transition with a minimum radius of 1 in. (25 mm) (AWS D1.6/D1.6M, clause 7.4.7.2). The reentrant corner may be formed by mechanical or thermal cutting, followed by grinding, if necessary, to meet the surface requirements of AWS D1.6/D1.6M, clause 7.4.5.

If dimensions of the part allow, the weld access hole should have a radius not less than ⅜ in. (10 mm).

Weld backing, if used, should comply with AWS D1.6/D1.6M, clause 7.9, and Table 7.1.

J2. WELDS

The AWS D1.6/D1.6M code contains the welding requirements for the fabrication, assembly, and erection of welded structures and weldments subject to design stress, where at least one of the materials being joined is stainless steel. It is intended for use with base metals with a minimum thickness of 1⁄16 in. (2 mm). The structural sections used in the welded structures covered by AWS D1.6/D1.6M are produced to the requirements of ASTM material or product specifications (see Section A3) with the exception of open welded sections made by methods other than laser or laser hybrid welding, where additional specification is needed by the engineer and, where necessary, testing (see Table A3.1).

AWS D1.6/D1.6M still references ASTM A167 (ASTM, 2009) even though this standard was withdrawn by ASTM some time ago. The correct specification is ASTM A240/A240M (ASTM, 2020f) (the relevant structural alloys had been removed from ASTM A167 well before it was withdrawn).

There are significant differences between welding stainless and carbon steels. Welding should only be done by stainless steel welders with current certification for the alloy and weld procedure. Furthermore, the significant differences between each stainless steel alloy family must be considered as well as any special considerations that might exist for a specific alloy.

The linear thermal expansion for austenitic stainless steels is approximately 30% higher than for carbon steels, and the thermal conductivity is about one-third that of carbon steel, so heat is conducted away from the weld zone more slowly. Both factors make shrinkage stresses greater, and both thin and thick plates deform easily. Therefore, control of distortion must be carefully considered when designing austenitic stainless steel welds. AWS D1.6/D1.6M requires careful sequencing of welds and procedures to limit distortion and drawings when weld distortion is of particular concern. In comparison, the thermal expansion of duplex stainless steels is only slightly higher than that of carbon steels, and the thermal conductivity is about half that of carbon steels so the heat is conducted away from the weld zone more slowly than for carbon steels. Because of these differences, special care is required for dissimilar metal welds between stainless and carbon steels.

Further information regarding the precautions to take when welding stainless steel is included in Section M2.6. Particular information on the welding of duplex stainless steel can be found in *Practical Guidelines for the Fabrication of Duplex Stainless Steels* (IMOA, 2014).

Although they are regularly welded, AWS D1.6/D1.6M does not have PWPS for all of the highly alloyed austenitic stainless steels or the duplex stainless steels, but Annex G provides nonmandatory guidelines for WPS qualification for these alloys. Annex G also provides guidance on welding of precipitation hardening stainless steels. Specific welding advice can be obtained from a stainless steel mill that produces the alloy or from a stainless steel welding products supplier. AWS D1.6/D1.6M, clause 5, provides the requirements for the generation and application of PWPS which are exempt from qualification by testing in accordance with clause 6. Otherwise, clause 6 provides the requirements for qualification of welding procedures and personnel.

1. Groove Welds

1a. Effective Area

Table J2.1 gives the same provisions as Table J2.1 in the 2016 AISC *Specification*. Table J2.2 is taken from AWS D1.6/D1.6M.

Partial-joint-penetration (PJP) groove welds joining the end of a hollow structural section (HSS) to a plate surface approximately perpendicular to the axis of the HSS, such as a base plate or cap plate, can be designed in the same manner as a non-HSS PJP weld. The effective throat is the shortest distance from the surface of the weld to the weld root. The weld root may not be at the joint root as this distance is dependent on the included angle, welding process, and welding position.

For PJP welds connecting two or more square or round HSS, see Chapter K and AWS D1.6/D1.6M, Figure 5.5.

1b. Limitations

The provisions were adopted from the 2016 AISC *Specification.*

2. Fillet Welds

2a. Effective Area

The provisions were adopted from the 2016 AISC *Specification.*

For skewed T-joints, the effective throat of a fillet weld is generally equal to the minimum thickness of the weld metal. When the skew angle is between 60° and 135°, this is the shortest distance from the surface to the joint root. When the skew angle is less than 60° but greater than 30°, the effective throat is reduced because of the inability to place filler metal at the back of the weld. See AWS D1.6/D1.6M, Annex B, for determining the effective throat. There are no prequalified fillet welds with a skew angle less than or equal to 30°, and welders need to qualify to weld joints with angles less than 30°. Frequently, welds with angles less than 30° are not included in the calculated strength of the connection.

The effective throat of a fillet weld does not include the weld reinforcement or any penetration beyond the weld root. Some welding procedures produce a consistent penetration beyond the root of the weld. This penetration contributes to the strength of the weld. However, it is necessary to demonstrate that the weld procedure to be used produces this increased penetration. In practice, this can be done initially by cross-sectioning the runoff tabs of the joint. Once this is done, no further testing is required, as long as the welding procedure is not changed.

2b. Limitations

Table J2.4 gives the minimum size of fillet welds and is taken from the 2016 AISC *Specification* and considered to be applicable to stainless steel also. Anything less than a ¼-in. (6-mm) leg fillet weld is difficult for a welder to achieve, although it is achievable with an automatic welding process.

The provisions for calculating the effective lengths of fillet welds are taken from the 2016 AISC *Specification*. There are no data for long stainless steel welds, but it is considered that the rules developed for carbon steel welds are applicable.

3. Plug and Slot Welds

3a. Effective Area

The provisions were adopted from the 2016 AISC *Specification.*

3b. Limitations

The provisions were adopted from the 2016 AISC *Specification.*

4. Strength

Similar to carbon steel welded joints, the strength of stainless steel welded joints is governed by the strength of either the base material or the deposited weld metal. Table J2.5 provides the nominal weld strengths and the ϕ and Ω factors for each weld type.

The provisions in this Specification for calculating the strength of groove, fillet, plug, and slot welded joints are the same as those included in the 2016 AISC *Specification* for carbon steel welded joints. However, the matching strength requirements for the filler metal given in the 2016 AISC *Specification* for CJP groove welded joints are not included in this Specification. This is because in contrast to carbon steel welds, where the filler metal is selected on the basis of its strength, for stainless steel welds the selection of the filler metal is predominantly based on metallurgical criteria. Although for prequalified austenitic welds, AWS D1.6/D1.6M requires the filler metal to have a strength that equals or exceeds the corresponding specified minimum base metal strength; for duplex welds or any other nonprequalified weld, there is no matching strength requirement imposed on the filler metal. For this reason, this Specification requires the strength of the weld metal to be evaluated when determining the strength of CJP welded joints. However, since the strength of the weld metal is based on the filler metal ultimate strength, F_{EXX}, and for CJP groove welded joints the strength of the base metal is given by the specified minimum yield strength, F_y, the strength of this type of welded joint will rarely be governed by the strength of the filler metal.

The resistance factors, ϕ, and the safety factors, Ω, given in Table J2.5 to determine the available strength of austenitic and duplex stainless steel welded joints are the same as those given in Table J2.5 of the 2016 AISC *Specification* for carbon steel welds, and less onerous than those recommended in the first edition of AISC Design Guide 27 (Baddoo, 2013), which were derived based on the limited amount of data on stainless steel welded joints available at the time of publication. The applicability of the resistance and safety factors adopted in this Specification for austenitic and duplex stainless steel welds was assessed based on the results from a reliability analysis carried out by Meza and Baddoo (2020) on austenitic and duplex stainless steel fillet welded joint specimens, and extrapolated to the other types of welded joints. The experimental data used in the reliability analysis were collected from research carried out by Lee et al. (2017), Yang et al. (2019), and Fortan et al. (2020) on fillet welded joint specimens tested with the load applied parallel or perpendicular to the weld axis. The nominal strength of the fillet welded joint specimens, R_n, was determined using Equation J2-5, which takes into account the angle between the line of action of the load and the longitudinal axis of the weld, θ. The data on stainless steel welded joints were divided into four groups with reference to the alloy family (austenitic or duplex) and the direction of the load with respect to the weld axis (parallel or perpendicular to the weld axis). The results from this reliability analysis showed that using a resistance factor, $\phi = 0.75$, and safety factor, $\Omega = 2.00$, for austenitic and duplex fillet welded joints lead to β-factors that ranged from 3.8 to 5.1, which were considered to be close enough to $\beta = 4$ targeted for connections in this Specification.

5. Combination of Welds

The provisions were adopted from the 2016 AISC *Specification.*

6. Welding Consumable and Electrode Requirements

The choice of filler metal should comply with the requirements for matching filler metals given in AWS D1.6/D1.6M, which states that the filler metal selection should predominantly be based on metallurgical criteria. For prequalified welds, AWS D1.6/D1.6M, clause 5.3.2, requires base metals to be welded with filler metals from either the corresponding alloy or a higher alloy filler metal in accordance with AWS D1.6/1.6M, Table 5.3. For nonprequalified welds, such as the higher alloyed austenitic and duplex stainless steels, selection of a lower strength filler metal might occur. AWS D1.6/D1.6M, Table 4.1, note b, requires the engineer to consider the strength of either overmatching or undermatching weld filler metal.

7. Mixed Weld Metal

The provisions were adopted from the 2016 AISC *Specification.*

8. Welding Dissimilar Steels

When welding stainless steel to carbon steel, the cleanliness of the surface, including complete removal of any zinc, is very important. Alloy dilution and type of filler metal are also critical. An over alloyed filler metal is used to compensate for the alloy dilution within the weld metal by the carbon steel.

When welding stainless steel to carbon steel, the provisions included in Section J2.8 require the paint system to extend a minimum distance of 2 in. (50 mm) beyond the weldment onto the stainless steel to avoid galvanic corrosion. This type of corrosion takes place when two dissimilar metals are in direct electrical contact and are also bridged by an electrolyte. Any moisture source—including condensation, humidity, rain, and immersion in liquids—can serve as an electrolyte. The chemistry of the liquid or deposits on the surface, such as chloride salts and pollutants, can increase the rate of corrosion. In this case, a current flows from the anodic metal (carbon steel) to the cathodic or nobler metal (stainless steel) through the electrolyte. As a result, the less noble metal corrodes.

Stainless steels usually form the cathode in a galvanic couple and, therefore, do not suffer additional corrosion. This form of corrosion is particularly relevant when considering joining stainless steel to carbon or low alloy steels or weathering steel. The filler metal is specified based on metallurgical criteria. Galvanic corrosion between different stainless steel alloys is hardly ever a concern, and then, only under fully immersed conditions.

J3. BOLTS AND THREADED PARTS

1. Stainless Steel Bolts

Whereas the RCSC *Specification* (RCSC, 2014) does not cover joints made with stainless steel bolts, many of its provisions are equally applicable to stainless steel as to carbon steel bolts and these provisions are listed in Section J3.1.

Although this Specification allows bolts with either unified national course (UNC) or 8 UN threads, resistance checks for bolts were developed based on UNC threads. The net tensile area of bolts with UNC threads is smaller than for bolts with 8 UN threads; therefore, it is conservative to base resistance checks on bolts having UNC threads.

There are four main ASTM standards covering stainless steel bolts. The first is for general applications, and the remaining three are used for specialized applications like low or high temperature, high strength, high pressure, and also when specifying bolt diameters over 1½ in. (38 mm).

ASTM F593, *Standard Specification for Stainless Steel Bolts, Hex Cap Screws, and Studs* (ASTM, 2017b)
This is the most common standard giving the chemical compositions and mechanical properties of smaller diameter [up to 1½ in. (38 mm)] stainless steel bolts for general corrosion resistance service applications. This standard includes austenitic, ferritic, martensitic, and precipitation hardening stainless steels for general corrosion resistance. Group numbers indicate that fasteners are chemically equivalent for general-purpose use. S30400/S30403 bolts are classified as Alloy Group 1, and S31600/S31603 are Alloy Group 2 bolts. The corresponding standard for nuts is ASTM F594, *Standard Specification for Stainless Steel Nuts* (ASTM, 2020d).

ASTM A320/A320M, *Standard Specification for Alloy-Steel and Stainless Steel Bolting for Low-Temperature Service* (ASTM, 2018b)
This standard covers austenitic and ferritic stainless steels and is intended specifically for low-temperature service, whether the application is structural or a piece of equipment. Type S30400 bolts are designated as B8 and B8A, and Type S31600 as B8M and B8MA. The standard covers bolts up to diameters of 1½ in. (38 mm). The corresponding standard for nuts is ASTM A962/A962M (ASTM, 2019d), *Standard Specification for Common Requirements for Bolting Intended for Use at any Temperature from Cryogenic to the Creep Range*.

ASTM A193/A193M, *Standard Specification for Alloy-Steel and Stainless Steel Bolting for High Temperature or High Pressure Service and Other Special Purpose Applications* (ASTM, 2020h)
This standard covers austenitic and ferritic stainless steel bolting for high temperature or high pressure service, or other special purpose applications. It includes both metric and U.S. customary units. This is the only standard that can be used for ordering stainless steel bolts in larger diameters. The corresponding standard for nuts is ASTM A194/A194M, *Standard Specification for Carbon Steel, Alloy Steel, and Stainless Steel Nuts for Bolts for High Pressure or High Temperature Service, or Both* (ASTM, 2020e).

ASTM A1082/A1082M *Standard Specification for High Strength Precipitation Hardening and Duplex Stainless Steel Bolting for Special Purpose Applications* (ASTM, 2016d)
This standard covers high-strength duplex and precipitation hardening stainless steels for special-purpose applications such as pressure vessels. Nuts are to be made from the stainless steels listed in the standard and tested to its requirements. This is the only ASTM standard that covers duplex stainless steel bolts, and it has no minimum or maximum size limit. The size limit for precipitation hardening stainless steels varies with the heat treatment condition but is generally 8 in. (200 mm).

If bolts are lubricated and have not been torqued to the point of galling, then reuse may be permitted with approval by the engineer of record for connections other than slip-critical joints and pretensioned connections subject to fatigue loading.

2. Size and Use of Holes

The provisions were adopted from the 2016 AISC *Specification*, except for the requirements for washers.

Washers are required for use under the bolt head because the research used to develop recommendations for slip-critical bolts was conducted on stainless steel bolts with hex heads, not heavy hex heads. Therefore, washers are needed under the bolt head to develop sufficient contact stress area.

3. Minimum Spacing

The provisions were adopted from the 2016 AISC *Specification*.

4. Minimum Edge Distance

The provisions were adopted from the 2016 AISC *Specification*.

5. Maximum Spacing and Edge Distance

Maximum criteria adopted in this Specification are the same as those given in the 2016 AISC *Specification*, and they are intended to eliminate local buckling of the plies.

6. Tension and Shear Strength of Bolts and Threaded Parts

Due to the variety of stainless steel bolts on the market, produced in accordance with different specifications with varying strength, this Specification does not give a table of nominal strengths for bolts in tension and shear. Instead, it gives expressions for F_{nt} *and* F_{nv} that were derived from a limited test program on M16 to M20 austenitic bolts subject to tension, shear, and combined tension and shear (SCI, 1990). Further tension and shear tests on individual bolt/nut assemblies were carried out under a European research program (European Commission, 2002). Some of the shear tests were carried out with the plates loaded in tension, and some in compression. Bolt diameters M12, M16, and M20 were tested; all the bolts were austenitic stainless steel bolts. Insufficient data were available to enable a reliability analysis to be carried out for tension rods and bolts in the same way as for austenitic and duplex stainless steels. Therefore, the appropriate resistance factors for austenitic and duplex stainless steel were reduced by 10% for precipitation hardening stainless steels to give an extra margin of safety.

7. Combined Tension and Shear in Bearing-Type Connections

The provisions for combined tension and shear in stainless steel bearing-type connections were adopted from the 2016 AISC *Specification*, with the exception of the ϕ and Ω values for precipitation hardening stainless steel bolts. These values are reduced from those of austenitic and duplex stainless steels as discussed in Commentary Section J3.6.

8. Stainless Steel Bolts in Slip-Critical Connections

Slip-critical connections are required when deformations in bolted connections must be limited to predefined values either for serviceability or ultimate limit reasons. Typical applications can be found in bridges, cranes, radio masts, and towers of wind turbines; for joints that are subject to fatigue load with reversal of the loading direction; or where functional requirements make slip-critical connections necessary (e.g., for sensitive machinery). Essential characteristics of these connections are firstly, the level of pretension in the bolts, and secondly, the slip coefficient, which is mainly influenced by the surface roughness of the clamped plates. For this reason, the level of pretension has to be guaranteed over the whole service life of the structure, and loss of pretensioning due to relaxation and creep effects (e.g., due to geometrical tolerances of the clamped plates or creep and relaxation of the structural elements themselves) has to be avoided.

In the past, stainless steel bolting assemblies were not used in slip-critical applications because of a lack of knowledge about their pretensioning behavior, friction coefficients, pretension losses, suitable tightening methods, etc. A European research project (European Commission, 2019; Afzali et al., 2017; Stranghöner et al., 2017a; Stranghöner et al., 2017b) was the first project to carry out a comprehensive study of the performance of austenitic and duplex stainless steel slip-resistant connections, and the provisions in this Specification are based on the results of this project. There are no product standards for stainless steel bolts specifically designed for pretensioning.

Tightening tests with stainless steel bolting assemblies were performed for varying bolt dimensions from M12 to M24 of austenitic and duplex bolts to study the pretensioning behavior. Bolts of two strengths were tested: the strength of the lower strength bolts was F_{yb} = 93 ksi (640 MPa) and F_{ub} = 116 ksi (800 MPa), and the strength of the stronger bolts was F_{yb} = 130 ksi (900 MPa) and F_{ub} = 145 ksi (1 000 MPa).

Section A3.3 requires hardened washers to be used under both the bolt head and nut in pretensioned joints subject to fatigue loading and slip-critical joints. Tests showed that washers should have at least 200 Brinell HBW for bolts with a tensile strength of 116 ksi (800 MPa) and 290 Brinell HBW for bolts with a tensile strength of 145 ksi (1 000 MPa) in order to limit the surface pressures on the clamped package and thus minimize pretension losses due to plastification of the surface. Embedding of the surface can be minimized by increasing the surface pressure area by using washers with a larger diameter than the nut or the head of the bolt. Depending on the yield strength of the clamped steel material, plastification of the surface of the clamped package can occur. For this reason, washers are more important for steel plates of lower strength. This Specification requires that all washers be hardened to 290 Brinell HBW, regardless of tensile strength.

The effects and importance of lubrication on the basic pretensioning behavior were studied. Galling can occur when bolting assemblies made of stainless steel are highly pretensioned and torqued. Experimental work showed that galling could be avoided up to a pretension level near the maximum individual bolt force when suitable lubricants are applied. All investigated bolting assemblies showed a tendency to severe galling when effective lubrication was not applied to the contact surfaces between the paired threads and the bearing surfaces between nut and washer.

Loss of pretension in stainless steel bolting assemblies may occur due to viscoplastic deformation behavior of the stainless steel bolting assembly and the clamped components. The viscoplastic deformation can be categorized in two groups: viscoplastic deformation under constant load condition for the plates (i.e., creep deformation) and viscoplastic deformation under constant strain condition for the bolts (i.e., stress relaxation). Stainless steel, as with other metals demonstrating nonlinear stress-strain characteristics, is known to creep at room temperature when loaded above its elastic limit (limit of proportionality). However, the studies on pretensioned bolting assemblies showed that pretension losses due to creep and relaxation effects are comparable to those of carbon steel.

This Specification requires that the suitability for pretensioning of a bolting assembly be certified by procedure testing. This procedure, known as a bolt tightening qualification procedure (BTQP), is presented in the second edition of AISC Design Guide 27 (Baddoo and Meza, 2021) and includes

- Bolt tension tests to confirm that the bolts meet the specifications in which they were ordered
- Suitability testing and analysis to determine key parameters of the fastener assembly installation behavior
- Evaluation of the suitability tests to establish if the fastener assemblies have sufficient strength, ductility, and lubrication

- Determination of tightening parameters such that fastener assemblies can reliably be installed to their specified pretension using one of the following installation methods: turn-of-nut, calibrated wrench, or combined

Equation J3-5 defines the minimum fastener tension, T_b, as 70% of the yield strength of the bolt multiplied by the net tensile area of the bolt, whereas the 2016 AISC *Specification* gives T_b as 70% of the tensile strength of the bolt multiplied by the net tensile area of the bolt. This is due to the tests showing that it was not always possible to guarantee a minimum fastener tension as high as 70% of the tensile strength of the bolt.

Table C-J3.1 gives the net tensile area of bolts.

The D_u value for stainless steel bolts is equal to 1.0 because only limited test data exist on the ratio of the mean installed bolt pretension to the specified minimum bolt pretension for stainless steel bolts. This is different from the D_u value of 1.13 for carbon steel bolts.

Slip coefficients for stainless steel faying surfaces were measured in accordance with the method in the European specification EN 1090-2 (CEN, 2018) for as-rolled, shot-blasted, and grit-blasted surfaces. Tests were subsequently carried out following the test procedures in RCSC *Specification* Appendix A, with adaptations made for testing bare stainless steel faying surfaces (Afzali et al., 2019). The tests showed that the asperity of grit-blasted faying surfaces is sharper than that of the shot-blasted surfaces and consequently provides better mechanical interlocking between the surfaces, which means better slip resistance behavior in the connections. This is the reason this Specification requires that all faying surfaces be grit blasted.

Many parameters influence the roughness of a blasted surface; for example, the type and size of the abrasive, angle, and pressure. For this Specification, the surface roughness, either *Rz* or *Rt*, was chosen to distinguish between surface classes and their associated slip coefficient values, as shown in Table M2.1. Because slip coefficient values for stainless steel are dependent on surface roughness, rather than surface condition as they are for carbon steels, definitions for the surface classes are provided in Section M2.13. These requirements specify that a steel fabricator must use a grit-blasting technique that achieves the desired surface roughness and also periodically inspect blasted surfaces during production to see that surface requirements are being met.

Currently, there are insufficient test data to define slip coefficients for surfaces smoother than Class SSB. The surface classes in Table J3.4 begin with Class SSB to align with the 2016 AISC *Specification* and to allow for the addition of Class SSA in a future edition of this Specification once there are sufficient slip coefficient test data on unblasted stainless steel faying surfaces.

This Specification does not provide slip coefficient values for unblasted faying surfaces because limited slip coefficient test data exist for these types of surfaces. Experimental tests have been conducted on unblasted faying surfaces meeting the requirements of ASTM A480/A480M (ASTM, 2020a), No. 1 Finish (Afzali et al., 2019). However, additional slip coefficient test data are needed because many other unblasted surface finish types exist for stainless steel.

TABLE C-J3.1
Net Tensile Area of Bolts

Bolt Diameter, d, in.	Net Tensile Area, A_s[a], in.^{2}	Threads per Inch, n[b]
5⁄8	0.226	11
3⁄4	0.334	10
7⁄8	0.462	9
1	0.606	8
1 1⁄8	0.763	7
1 1⁄4	0.969	7
1 3⁄8	1.16	6
1 1⁄2	1.41	6

[a] Net tensile area = $(\pi/4)(d - 0.9743/n)^2$
[b] For diameters listed, thread series is UNC (coarse).

Blasting the stainless steel surfaces increases the possibility of corrosion as small abrasive media may become embedded in the surfaces. Only the faying surfaces of the slip-critical connection have to be blasted.

Slip-critical connections comprising stainless steel bolts and carbon steel plate are outside the scope of this Specification because of a lack of test data. It is standard practice to isolate dissimilar metals in bearing-type connections using compressible washers and bushings. For a slip-critical connection, it would be necessary to use incompressible materials.

Note also that welding the nut to the bolt to prevent the former from unscrewing is a practice to be avoided. A second nut can be added if necessary to prevent the bolt from loosening.

10. Bearing and Tearout Strength at Bolt Holes

Provisions for bearing strength of pins differ from those for bearing strength of bolts; refer to Section J8.

Bearing strength values are provided as a measure of the strength of the material upon which a bolt bears, not as a protection to the fastener, which needs no such protection. Accordingly, the same bearing value applies to all joints assembled by bolts, regardless of fastener shear strength or the presence or absence of threads in the bearing area.

Material bearing strength may be limited either by bearing deformation of the hole or by tearout (a bolt-by-bolt block shear rupture) of the material upon which the bolt bears.

The bearing strength of stainless steel connections has been investigated by Salih et al. (2011). Whereas the load-deformation curve for carbon steel connections flattens off after the initiation and spreading of yielding, for stainless steel connections this curve continues to rise significantly owing to strain hardening. For this reason, greater clarity in defining bearing capacity than has previously been used when considering carbon steel connections was necessary. Different failure definitions were devised for stainless steel connections, and bearing design equations for both thick and thin material that cover two cases (one restricting and one ignoring serviceability deformations) were proposed. The recommendations for thick material given in Salih et al. (2011) have been included in Section J3.10, which are based on numerical analysis. For bolted connections where deformation is not a key design consideration, the design formula was proposed based on the numerically predicted ultimate bearing resistance taken as the maximum attained load regardless of the associated deformation (i.e., similar to the treatment for net section failure). With regards to bolted connections where deformation is a design consideration, the design expression was developed on the basis of the numerically predicted ultimate bearing resistance taken as the load at a prespecified acceptable deformation, with an adopted value of $1/32$ in. (1 mm) under serviceability conditions for stainless steel connections.

The accuracy of the design formulae was assessed through comparisons against the test results from Cai and Young (2014). The comparisons were made based on the measured material and geometric properties from the test specimens, and with all safety factors set equal to unity. The proposed design formulae were shown to yield accurate and consistent bearing resistance predictions of bolted connections, with the mean test-to-predicted resistance ratio equal to 1.14 and a coefficient of variation of 0.09.

12. Wall Strength at Tension Fasteners

With any connection configuration where the fasteners transmit a tensile force to the HSS wall, a rational analysis must be used to determine the appropriate limit states. These may include a yield-line mechanism in the HSS wall and/or pull-out through the HSS wall, in addition to applicable limit states for the fasteners subject to tension.

13. Bolting Dissimilar Metals

The design of joints needs careful attention to maintain optimum corrosion resistance. This is especially true for joints that may become wet from the weather, spray, immersion, or condensation, etc. Further guidance is given in AISC Design Guide 27 on galvanic corrosion and design detailing to avoid it.

The use of carbon steel bolts to connect stainless steel elements, whether or not the bolts are galvanized, is prohibited because the bolts will corrode at a highly accelerated rate relative to the stainless steel due to the potential difference and adverse surface area difference. The rate of crevice corrosion will also be accelerated by the potential difference.

Slip-critical connections in which stainless steel bolts connect carbon steel plates are outside the scope of the design provisions in this Specification. However, the performance of these types of connections could be verified by testing, although it would be necessary for the metals to be galvanically isolated if the conditions for galvanic corrosion exist.

J4. AFFECTED ELEMENTS OF MEMBERS AND CONNECTING ELEMENTS

The provisions in this section were adopted from the 2016 AISC *Specification* except as detailed in this Commentary.

2. Strength of Elements in Shear

The value of the resistance factor for shear yielding is 0.90 (with an equivalent safety factor of 1.67), which is consistent with the provisions in Chapter G. This is different from the value for resistance factor of 1.00 in the 2016 AISC *Specification* (with an equivalent safety factor of 1.50). Prior to the 2005 AISC *Specification* (AISC, 2005), a resistance factor of 0.90 was used. After that it was increased to 1.00 to align with the ASD specification—the increase in LRFD design strength being justified by the long history of satisfactory performance of ASD use.

J5. FILLERS

The provisions in this section were adopted without modification from the 2016 AISC *Specification*, except for the requirement that the filler material should have equivalent corrosion resistance and strength to that of the structural member. Background information is given in the 2016 AISC *Specification* Commentary.

J7. BEARING STRENGTH

The provisions in this section were adopted without modification from the 2016 AISC *Specification*, except that no provisions are given in this Specification for expansion rollers and rockers.

In general, the bearing strength design of finished surfaces is governed by the limit state of bearing (local compressive yielding) at nominal loads. The nominal bearing strength of milled contact surfaces exceeds the yield strength because adequate safety is provided by post-yield strength as deformation increases.

J8. PINS

The provisions in Section J8 address the design of pins in pin-connected assemblies typically used for structural applications.

The methodology of calculating the moment in a pin by assuming it is simply supported by the connected parts is because in most pin-connected assemblies in structural applications a diametral clearance exists between the pin and the connecting parts. Due to this clearance, as the assembly is tensioned, the pin bends and contact between the pin and the connecting parts localizes at small regions. Figure C-J8.1 shows a typical pin-connected assembly under load, P, for which the reaction forces in the pin can be simplified as four concentrated forces.

Due to the ductility of stainless steel, localized deformations take place on the surface of the pin and connecting parts at the points of contact, which result in the reaction forces being transmitted into the pin through an effective contact patch, rather than a theoretical point, as shown in Figure C-J8.2. Numerical simulations on a large number of pin-connected assemblies have shown that the dimensions of the contact patches can be approximated as

$$h = 0.41 d_{pin} \quad \text{(C-J8-1)}$$

$$L_A = \frac{P}{1.8hF_{y,Pin\text{-}A}} \quad \text{(C-J8-2)}$$

$$L_B = \frac{P}{1.8hF_{y,Pin\text{-}B}} \quad \text{(C-J8-3)}$$

where $F_{y,Pin\text{-}A}$ is the lesser between the specified minimun yield strength of the pin and connecting part A, $F_{y,Pin\text{-}B}$ is the lesser between the specified minimum yield strength of the pin and connecting part B, and d_{pin} is the diameter of the pin. The maximum bending moment in the pin can thus be calculated using Equation C-J8-4. A similar approach can be used to calculate the maximum bending moment in a pin in contact with a greater number of connecting parts:

$$M_{pin,\max} = \frac{P\left(2c + L_A + L_B\right)}{4} \quad \text{(C-J8-4)}$$

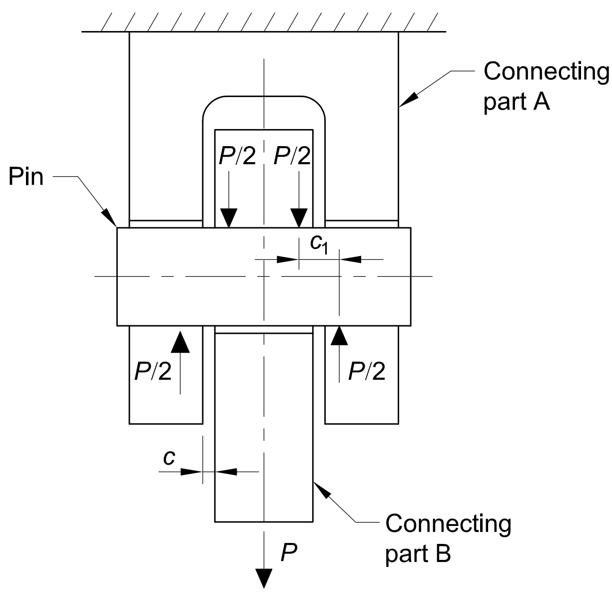


Fig. C-J8.1. Typical pin-connected assembly.

J9. COLUMNS BASES AND BEARING ON CONCRETE

The provisions in this section were adopted without modification from the 2016 AISC *Specification*. Background information is given in the Commentary to that standard.

J10. ANCHOR RODS AND EMBEDMENTS

The term "anchor rod" is used for threaded rods embedded in concrete to anchor structural steel. The term "rod" is intended to clearly indicate that these are threaded rods, not structural bolts, and should be designed as threaded parts in accordance with Section J3.6 using the material specified in Section A3.4.

Generally, the largest tensile force for which anchor rods must be designed is that produced by bending moment at the column base and augmented by any uplift caused by the overturning tendency of a building under lateral load.

J11. DOUBLY SYMMETRIC I-SHAPED MEMBERS WITH CONCENTRATED FORCES

The provisions in this section have been adopted from the 2016 AISC *Specification* without modification. They are known to be conservative; however, it was decided to retain them until research on stainless steel I-shaped members subject to concentrated forces justify the adoption of more accurate design rules.

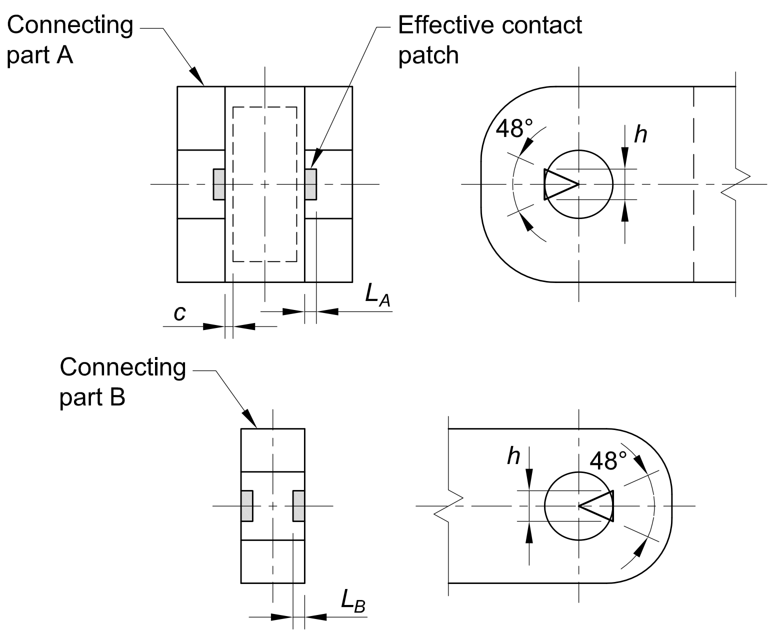


Fig. C-J8.2. Effective contact patches.

J12. SQUARE AND RECTANGULAR HSS WITH CONCENTRATED FORCES

The provisions in this section are specific to stainless steel square and rectangular hollow structural sections (HSS). The four specified loading cases given in Table J12.1—namely, end one-flange loading (EOF), interior one-flange loading (IOF), end two-flange loading (ETF), and interior two-flange loading (ITF)—are in line with those in the *North American Specification for the Design of Cold-Formed Steel Structural Members* (AISI, 2016a).

The behavior of cold-formed steel sections under locally distributed edge forces has been investigated by many researchers. It has been found that the theoretical analysis of web crippling for square and rectangular HSS under concentrated bearing forces is rather complicated because it involves numerous factors, such as (a) nonuniform stress distribution under the applied force and adjacent portions of the web, (b) elastic and inelastic stability of the web element, (c) local yielding in the immediate region of load application, (d) bending produced by eccentric load or reaction when it is applied on the bearing flange at a distance beyond the curved transition of the web, (e) initial out-of-plane imperfection of plate elements, and (f) various edge restraints provided by beam flanges and interaction between flange and web elements (AISI, 2016b). For these reasons, the web crippling provisions in Section J12 are based upon the available experimental data on stainless steel square and rectangular HSS reported by Talja and Salmi (1995), Gardner et al. (2006), Zhou and Young (2006, 2007), Talja and Hradil (2012), Islam and Young (2012, 2014), and Cai and Young (2019a, 2019b), as summarized in Table C-J12.1.

Prabakaran (1993) and Prabakaran and Schuster (1998) developed a unified equation (Equation J12-1) for various cold-formed steel sections under different loading cases. This equation has been adopted in this section for stainless steel square and rectangular HSS. The newly calibrated coefficients are summarized in Table J12.1 of the Specification, and the parametric limitations given are based on the experimental data. The nominal strength, R_n, in Section J12 applies to the entire square and rectangular HSS, not only for a single web.

Table C-J12.1
Data Used for Calibration of Design Provisions

References	Materials*	Number of Tests			
		EOF	IOF	ETF	ITF
Talja & Salmi (1995)	A	–	6	–	–
Gardner et al. (2006)	HSA	–	6	–	–
Zhou & Young (2006)	A	–	–	17	16
Zhou & Young (2007)	HSA & D	11	14	15	15
Islam & Young (2014)	LD	3	3	5	5
Cai & Young (2019a)	LD	8	–	19	–
Cai & Young (2019b)	LD	–	16	-	21

* Note: A = austenitic stainless steel; HSA = high-strength austenitic stainless steel; D = duplex stainless steel; LD = lean-duplex stainless steel.

CHAPTER K

ADDITIONAL REQUIREMENTS FOR HSS AND BOX-SECTION CONNECTIONS

Chapter K addresses the strength of simple connections to austenitic and duplex stainless steel square or round hollow structural sections (HSS) and box sections of uniform wall thickness, where seam welds between box-section elements are complete-joint-penetration groove welds in the connection region.

Compared to the 2016 AISC *Specification*, this Specification does not include provisions for plate-to-HSS connections, moment connections, or rectangular HSS-to-HSS connections, due to a lack of experimental data on these types of stainless steel HSS connections.

K1. GENERAL PROVISIONS

The provisions of this chapter are based on the CIDECT (Wardenier et al., 1991; Packer et al., 1992) design rules for welded carbon steel HSS joints, which have also been adopted by the 2016 AISC *Specification*. Tests reported by Rasmussen and Young (2001) and Rasmussen and Hasham (2001) on welded X- and K-joints in square and circular hollow sections showed that these provisions can be safely applied to compute the resistance of the types of stainless steel connections included in Section K2. These test data, which only include austenitic stainless steel connections, were previously used by Rasmussen and Young (1994) to carry out a reliability analysis that showed that a resistance factor of 0.90 can be used with the CIDECT strength equations. Because no data are available for duplex stainless steel, a resistance factor of 0.90 was adopted, based on the findings for austenitic stainless steel, which demonstrate greater material nonlinearity.

The 2016 AISC *Specification* limited the applicability of the CIDECT recommendations to carbon steel with a yield stress of up to 51 ksi (355 MPa). This limit was imposed partly because carbon steel joints with yield stresses greater than 51 ksi (355 MPa) may not have adequate ductility. Since both austenitic and duplex stainless steels generally have high ratios of tensile strength to 0.2% offset yield strength and high values of elongation after fracture, no strength limitation was imposed for stainless steel HSS connections.

K2. HSS-TO-HSS TRUSS CONNECTIONS

The design rules in this section apply to geometries in which the centerlines of the branch member(s) and the chord members lie in a single plane. In addition, in order for the connection to be treated as a truss connection in which the members are connected by pinned joints, eccentricity between the centerlines of the branch members and the chord member should lie within the limits given in Table K2.1a or Table K2.2a.

The provisions in this chapter for the design of welded HSS connections are based on potential strength limit states that may arise for particular connection geometries and loading, which, in turn, represent possible failure modes that may occur within prescribed limits of applicability. There is no connection deformation limit state considered in these provisions. While for carbon steel, the CIDECT strength expressions and their limit of validity implicitly guarantee that at service loads deformations are below the limit of 1% of the chord width or diameter, the increased nonlinearity exhibited by stainless steel makes it more susceptible to serviceability issues, and there may be some cases in which the serviceability limit may be exceeded, as described in the following Commentary.

Chapter K does not prohibit using joints that fall outside the listed limits of applicability; however, this Specification and Commentary do not provide connection capacities or guidance when doing so. A rational approach to their design is left to the designer. Further information regarding the failure modes that should be considered when designing connections that fall outside the limits of applicability given in this section can be found in the Commentary to the 2016 AISC *Specification*.

The 2016 AISC *Specification* Commentary Section K1 provides a definition of K-, T-, and Y-connection configurations.

2. Round HSS

The tests used to assess the applicability of the CIDECT strength equations to stainless steel round HSS-to-HSS truss connections only included cross- and K-joints (Rasmussen and Hasham, 2001). However, given the consistently conservative test strengths reported from these experiments, it was deemed safe to extend the applicability range of the CIDECT strength equations to cover T- and Y-connections also, as given in Table K2.1.

Although the experiments carried out by Rasmussen and Hasham (2001) showed that the CIDECT equations provided conservative strength predictions for all the tested joints, the deformation of one of the cross-connection specimens exceeded the 1% deformation limit at service loads. In order to address this issue, a User Note was added to warn engineers that if limiting deformations is a critical part of the design, a more detailed analysis should be carried out for these types of joints.

3. Square HSS

Although the tests used to assess the applicability of the CIDECT strength equations to stainless steel square HSS-to-HSS truss connections included cross- and K-connections and for both types of connections the CIDECT strength equations were found to yield conservative predictions, this Specification only includes K-connections to align with the types of connections covered by the 2016 AISC *Specification*.

Tests on stainless steel square HSS K-connections reported by Rasmussen and Young (2001) showed that the serviceability deformation limit of 1% of the chord width was not exceeded if they fall within the limits of applicability given by Table K2.2A; therefore, for this type of connection it is not necessary to check joint deformations under service loads.

CHAPTER L
DESIGN FOR SERVICEABILITY

L1. GENERAL PROVISIONS

Serviceability may need to be checked under various service loads, including (a) static loads from the occupants, snow or rain on the roof, or temperature fluctuations and (b) dynamic loads from human activities, wind effects, the operation of mechanical or building service equipment, or traffic near the building. Service loads are loads that act on the structure at an arbitrary point-in-time and may be only a fraction of the corresponding nominal load. The response of the structure to service loads generally can be analyzed assuming elastic behavior. Members that accumulate residual deformations under service loads also may require examination with respect to this long-term behavior.

Serviceability limit states and appropriate load combinations for checking conformance to serviceability requirements can be found in ASCE/SEI 7, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, Section 1.3.2, Commentary Section 1.3.2, Appendix C, and Commentary Appendix C (ASCE, 2016).

The Commentary to the 2016 AISC *Specification* gives further background information.

L2. DEFLECTIONS

Excessive vertical deflections and misalignment arise primarily from three sources: (a) gravity loads, such as dead, live, and snow loads; (b) effects of temperature, creep, and differential settlement; and (c) construction tolerances and errors. Appropriate limiting values of deformations depend on the type of structure, detailing, and intended use (Galambos and Ellingwood, 1986). Historically, common deflection limits for horizontal members have been $1/360$ of the span for floors subjected to reduced live load and $1/240$ of the span for roof members. Deflections of about $1/300$ of the span (for cantilevers, $1/150$ of the length) are visible and may lead to general architectural damage or cladding leakage. Deflections greater than $1/200$ of the span may impair operation of movable components such as doors, windows, and sliding partitions.

Proper control of deflections is a complex subject requiring careful application of professional judgment. AISC Design Guide 3, *Serviceability Design Considerations for Steel Buildings*, 2nd Edition (West et al., 2003), provides an extensive discussion of the issues. Further information about deflection can also be found in the Commentary to the 2016 AISC *Specification*, including recommended load combinations for computing deflections.

All steels with gradual yielding show transient creep when loaded above their elastic limit (the limit of proportionality) and the amount of creep deformation increases with increasing load level. The elastic limit for austenitic and duplex stainless steels is around 60% of the strength at the 0.2% offset permanent strain. For members subject to long-term loading at stresses close to the 0.2% offset strength, creep deformations should be taken into consideration (e.g., pretensioned strands). The creep and relaxation behavior of austenitic and duplex stainless steels was characterized in a recent European research project (European Commission, 2019).

Table User Note L2.1 gives the secant modulus for common types of stainless steel at a maximum stress in the cross section equal to $0.6F_y$. Whereas the compressive stress in doubly symmetric cross sections is the same as the tensile stress, this is not the case for asymmetric cross sections; therefore, using these approximate values can be exceedingly conservative for asymmetric cross sections.

L3. DRIFT

Drift limits are imposed on buildings to minimize damage to cladding and to nonstructural walls and partitions. Lateral frame deflection is evaluated for the building as a whole, where the applicable parameter is the total building drift, defined as the lateral frame deflection, Δ, at the top of the most occupied floor divided by the height of the building to that level, H, that is, Δ/H. For each floor, the applicable parameter is interstory drift, defined as the lateral deflection of a floor, δ_n, relative to the lateral deflection of the floor immediately below, δ_{n-1}, divided by the distance between floors, h; that is, $\left(\delta_n - \delta_{n-1}\right)/h$.

Typical drift limits in common usage vary from $H/100$ to $H/600$ for total building drift and $h/200$ to $h/600$ for interstory drift, depending on building type and the type of cladding or partition materials used. The most widely used values are H (or h)/400 to H (or h)/500 (ASCE, 1988). These limits generally are sufficient to minimize damage to cladding and nonstructural walls and partitions. Smaller drift limits may be appropriate if the cladding is brittle. AISC Design Guide 3 (West et al., 2003) contains recommendations for higher drift limits that have successfully been used in low-rise buildings with various cladding types.

Further information regarding the importance of drift control to prevent damage to building elements can be found in the Commentary to the 2016 AISC *Specification*.

L4. VIBRATION

An extensive treatment of vibration in steel-framed floor systems and pedestrian bridges is found in AISC Design Guide 11, *Vibrations of Steel-Framed Structural Systems Due to Human Activity* (Murray et al., 2016). This guide provides basic principles and simple analytical tools to evaluate steel-framed floor systems and footbridges for vibration serviceability due to human activities, including walking and rhythmic activities.

L5. WIND-INDUCED MOTION

Acceleration in wind-sensitive buildings may be expressed as either root mean square (RMS) or peak acceleration. Both measures are used in practice, and there is no clear agreement as to which is the more appropriate measure of motion perception. Guidance for design acceleration levels used in building design may be found in the literature (Chen and Robertson, 1972; Hansen et al., 1973; Irwin, 1986; NRCC, 1990; Griffis, 1993).

Further information regarding wind-induced motion can be found in the Commentary to the 2016 AISC *Specification*.

L6. THERMAL EXPANSION AND CONTRACTION

The satisfactory accommodation of thermal expansion and contraction cannot be reduced to a few simple rules but must depend largely upon the judgment of a qualified engineer. Conditions during construction, such as temperature effects before enclosure of the structure, should also be considered. Complete separation of the framing at widely spaced expansion joints is generally more satisfactory than more frequently located devices that depend upon the sliding of parts in bearing. Guidelines for the recommended size and spacing of expansion joints in buildings may be found in NRC (1974).

Note that the coefficient of thermal expansion for austenitic stainless steels is about 30% higher than that of carbon steel. Where carbon steel and austenitic stainless steel are used together, the effects of differential thermal expansion should be considered in design. The thermal expansion of duplex stainless steel is very similar to that of carbon steel. The thermal conductivity of austenitic and duplex stainless steels is about 30% of that of carbon steel.

Table User Note A3.1 gives the values of the coefficient of thermal expansion of austenitic and duplex stainless steel for temperatures ranging between 68°F (20°C) and 212°F (100°C).

L7. CONNECTION SLIP

In bolted connections with bolts in holes having only small clearances, such as standard holes and slotted holes loaded transversely to the axis of the slot, the amount of possible slip is small. Slip at these connections is not likely to have serviceability implications. Possible exceptions include certain unusual situations where the effect of slip is magnified by the configuration of the structure, such as a connection at the base of a shallow cantilever beam or post where a small amount of bolt slip may produce unacceptable rotation and deflection.

This Specification requires that connections with oversized holes or slotted holes loaded parallel to the axis of the slot be designed as slip-critical connections. For a discussion of slip at these connections, see Commentary Section J3.8. Where slip at service loads is a realistic possibility in these connections, the effect of connection slip on the serviceability of the structure must be considered.

CHAPTER M
FABRICATION AND ERECTION

M1. FABRICATION AND ERECTION DOCUMENTS

Significant guidance on the fabrication and erection of stainless steel is readily available in the following industry association brochures that, although not legally binding, are often referenced in fabrication and erection documents:

Practical Guidelines for the Fabrication of Duplex Stainless Steels (IMOA, 2014)

The Forming Potential of Stainless Steel (Euro Inox, 2006a)

Erection and Installation of Stainless Steel Components (Euro Inox, 2006b)

Bending Stainless Steel Tube—Design Benefits in Engineering and Architecture (Euro Inox, 2012a)

Stainless Steels: Tables of Fabrication Parameters (Euro Inox, 2012b)

Fabrication and Post-Fabrication Cleanup of Stainless Steels (Tuthill, 1986)

Electroforming—A Unique Metal Fabrication Process (NIDI, 1998)

Practical Guidelines for the Fabrication of High Performance Austenitic Stainless Steels (IMOA, 2010)

Practical Guide to Using 6Mo Austenitic Stainless Steel (NIDI, 1994)

High Performance Stainless Steels (NIDI, 2000)

Practical Guidelines for the Fabrication of Austenitic Stainless Steels (IMOA, 2020)

M2. FABRICATION

1. Identification

Hard stamps, punches, and drill marks are not allowed for stainless steel because they are stress risers. Also, these markings may be aesthetically objectionable.

It is critical to remove temporary color coding or marking with paint, crayon, ink stencils, or similar approved chloride- and sulfide-free marking products prior to project completion because they are well-known sources of corrosion problems if left on the stainless steel during service. These contaminants can act as crevices and initiation sites for pitting and crevice corrosion. Additionally, if they are present in an area where welding occurs, they can cause carbide precipitation, which can lead to sensitization and intergranular corrosion during service.

2. Handling and Storage

Magnetic lifts are not used for austenitic or duplex stainless steels. Austenitic stainless steels have very low magnetic permeability and, with a partially austenitic microstructure, duplex stainless steels have a substantially lower magnetic permeability than carbon steels.

Suitable handling and storage measures to see that the surface finish of stainless steel is not damaged include, but are not limited to:

(a) Use of protective strippable film, paper interleave, plastic wrapping, or other suitable protective packaging, to be left on as long as practical. If strippable film is applied, it should be on the surface no longer than the period of the film warranty and not exposed to temperatures outside the manufacturer's approved exposure range. If the location is exposed to salt spray or mist, the film should be removed as soon as possible to prevent crevice corrosion under the film. Only ultraviolet rated film should be exposed to sunlight.

(b) Avoidance of unprotected storage or transportation in locations exposed to salt-laden coastal atmospheres or deicing salt mist. Precautions should also be taken to avoid immersion in seawater, as can occur during a storm surge.

(c) Protection of storage racks by wooden, rubber, or plastic battens or sheaths to avoid free iron contamination from free iron, copper, zinc, or other metallic contact with stainless steel surfaces.

(d) Protection of stainless steel from direct contact with other steel alloy lifting tackle, temporary supports or handling equipment such as chains, hooks, strapping, and rollers or the forks of fork lift trucks by use of isolating materials or light plywood or suction cups. Appropriate erection tools should be used such that surface contamination does not occur.

(e) Avoidance of contact with chemicals and acids, including dyes, glues, adhesive tape, hydrochloric acid, chloride containing cleaning products, and undue amounts of oil and grease. If it is necessary to use them, their suitability and the maximum duration of exposure should be determined.

(f) Use of segregated manufacturing areas for stainless steel to prevent free iron or other metals from being embedded in the surface.

(g) Use of separate tools dedicated for use on stainless steel only, particularly grinding wheels and wire brushes. Metal wire brushes and wire wool used on stainless steel should be stainless steel, preferably an austenitic stainless steel of equivalent corrosion resistance.

Most stocked stainless steel brushes and wool are either the martensitic stainless steel S41000 (410) or the austenitic stainless steel S30200 (302). S41000 is significantly less corrosion resistant than any of the stainless steels referenced in this Specification. S30200 would be acceptable for the less corrosion-resistant stainless steels in this Specification, such as the precipitation hardening stainless steels or S30400/S30403 (304/304L). If a low alloy stainless steel brush or wire wool is used on a higher alloy stainless steel, particles of the lower alloyed stainless steel may embed in the surface and cause unanticipated corrosion if the service environment is too corrosive for the lower alloy stainless steel. Wire brushes that are made from stainless steels that match the corrosion resistance of higher alloyed stainless steels may require special manufacturing and incur large costs and delay. For higher alloy stainless steels, the use of flexible nonwoven abrasive discs may be preferred over wire brushing.

3. Cambering, Curving, and Straightening

Cold bending, curving, drawing, and other forming of austenitic and duplex stainless steels is common. Power requirements for bending stainless steel are higher than for bending geometrically similar components of other steel alloys. Both springback and work hardening must be considered. Work hardening must be considered when forming austenitic stainless steels as their strength can increase by about 50%. Hardness testing can be an effective means of determining that there has not been excessive cold working during straightening or curving. The amount of springback that will occur after bending will vary with the alloy family, specific alloy, and (to a lesser degree) the chemistry variation of the heat; reference charts are available, but some tooling adjustment may be required. The springback for duplex stainless steels is greater than for austenitic stainless steels. Nitrogen alloying additions are used to increase strength (and corrosion resistance) and have a significant effect on behavior. The elastic springback of an alloy determines the amount of overbending that is required to achieve the desired tolerances, and a high level of springback may make forming tighter bends impossible and preclude the use of a specific alloy in some designs. Higher strength stainless steels, such as the duplex and austenitic stainless steels alloyed with nitrogen, require higher forming forces and show more elastic springback.

4. Cutting

Oxyacetylene torch cutting can only be used for cutting metals whose oxides have a low melting point, like low- to medium-carbon steels and wrought iron. Metals like stainless steel cannot be cut due to the formation of a chromium oxide that prevents oxidation from fully occurring.

6. Welded Construction

Regarding back-side shielding, oxidation of the back side of welds can cause “sugaring” which has an unattractive appearance and reduced corrosion resistance. Back shielding of duplex stainless welds with nitrogen deters the loss of nitrogen at the weld root and helps maintain sufficient austenite needed for ductility and corrosion resistance. Back shielding can minimize the formation of heat tint and facilitate cleaning per ASTM A380/A380M (ASTM, 2017c); therefore, back shielding should be used whenever the possibility of exposure of the weld root exists. Back shielding should be considered whenever there is a possibility of significant heating of the back side of the material being welded.

CHAPTER N

QUALITY CONTROL AND QUALITY ASSURANCE

This chapter on quality control and quality assurance does not address a number of applications associated with structural stainless steel. The following is a list of references that may help with quality control and quality assurance for some of these items:

(1) Steel (open web) joists and joist girders—Each model specification of the Steel Joist Institute contains a section on quality.

(2) Concrete reinforcing bars, concrete materials, or placement of concrete for composite members—ACI 318 and ACI 318M (ACI, 2014).

N1. GENERAL PROVISIONS

This chapter provides minimum requirements for quality control (QC), quality assurance (QA), and nondestructive testing (NDT) for structural stainless steel systems for buildings and other structures.

This chapter also defines a comprehensive system of quality control requirements on the part of the stainless steel fabricator and erector and similar requirements for quality assurance on the part of the project owner's representatives when such is deemed necessary to complement the contractor's quality control function. These requirements exemplify recognized principles of developing involvement of all levels of management and the workforce in the quality control process as the most effective method of achieving quality in the constructed product. The chapter supplements these quality control requirements with quality assurance responsibilities as are deemed suitable for a specific task. The requirements follow the same requirements for inspections utilized in AWS D1.6/D1.6M (AWS, 2017) and the RCSC *Specification* (RCSC, 2014).

Under AISC *Code of Standard Practice for Structural Stainless Steel Buildings* Section 8 (AISC, 2021), the fabricator or erector is to implement a QC system as part of their normal operations. Those that participate in AISC Quality Certification or similar programs are required to develop QC systems as part of those programs. The engineer of record (EOR) should evaluate what is already a part of the fabricator's or erector's QC system in determining the quality assurance needs for each project. Where the fabricator's or erector's QC system is considered adequate for the project, including compliance with any specific project needs, the special inspection or QA plan may be modified to reflect this. Similarly, where additional needs are identified, supplementary requirements should be specified.

The terminology adopted is intended to provide a clear distinction between fabricator and erector requirements and the requirements of others. The definitions of QC and QA used here are consistent with usage in related industries, such as the steel bridge industry, and they are used for the purposes of this Specification. It is recognized that these definitions are not the only definitions in use. For example, QC and QA are defined differently in the AISC Quality Certification program in a fashion that is useful to that program and are consistent with the International Standards Organization and the American Society for Quality.

For the purposes of this Specification, QC includes those tasks performed by the stainless steel fabricator and erector that have an effect on quality or are performed to measure or confirm quality. QA tasks performed by organizations other than the stainless steel fabricator and erector are intended to provide a level of assurance that the product meets the project requirements.

The terms quality control and quality assurance are used throughout this Chapter to describe inspection tasks required to be performed by the stainless steel fabricator and erector and project owner's representatives, respectively. The QA tasks are inspections often performed when required by the applicable building code or authority having jurisdiction (AHJ), and designated as "special inspections," or as otherwise required by the project owner or engineer of record.

Chapter N defines two inspection levels for required inspection tasks and labels them as either "observe" or "perform." The choice in terminology reflects the multi-task nature of welding and bolting operations, and the required inspections during each specific phase.

N2. FABRICATOR AND ERECTOR QUALITY CONTROL PROGRAM

Many quality requirements are common from project to project. Many of the processes used to produce structural stainless steel have an effect on quality and are fundamental and integral to the fabricator's or erector's success. Consistency in imposing quality requirements between projects facilitates more efficient procedures for both.

The construction documents referred to in this chapter are, of necessity, the versions of the design documents, specifications, and approval documents that have been released for construction, as defined in the AISC *Code of Standard Practice for Structural Stainless Steel Buildings* (AISC, 2021). When responses to requests for information and change orders exist that modify the construction documents, these also are part of the construction documents. When a building information model is used on the project, it also is a part of the construction documents.

Elements of a quality control program can include a variety of documentation, such as policies, internal qualification requirements, and methods of tracking production progress. Any procedure that is not apparent subsequent to the performance of the work should be considered important enough to be part of the written procedures. Any documents and procedures made available to the quality assurance inspector (QAI) should be considered proprietary and not distributed inappropriately.

The inspection documentation should include the following information:

(1) The product inspected

(2) The inspection that was conducted

(3) The name of the inspector and the time period within which the inspection was conducted

(4) Nonconformances and corrections implemented

Records can include marks on pieces, notes on drawings, process paperwork, or electronic files. A record showing adherence to a sampling plan for pre-welding compliance during a given time period may be sufficient for pre-welding observation inspection.

The level of detail recorded should result in confidence that the product is in compliance with the requirements.

N3. FABRICATOR AND ERECTOR DOCUMENTS

1. Submittals for Stainless Steel Construction

The documents listed must be submitted so that the EOR or the EOR's designee can evaluate that the items prepared by the fabricator or erector meet the EOR's design intent. This is usually done through the submittal of approval documents. For additional information concerning this process, refer to the AISC *Code of Standard Practice for Structural Stainless Steel Buildings* (AISC, 2021).

2. Available Documents for Stainless Steel Construction

The documents listed must be available for review by the EOR. Certain items are of a nature that submittal of substantial volumes of documentation is not practical; therefore it is acceptable to have these documents reviewed at the fabricator's or erector's facility by the engineer or designee, such as the QA agency. Additional commentary on some of the documentation listed in this section follows:

(1) This section requires documentation to be available for the fastening of deck. For deck fasteners, such as screws and power fasteners, catalog cuts and/or manufacturers installation instructions are to be available for review. There is no requirement for certification of any deck fastening products.

(2) Because the selection and proper use of welding filler metals is critical to achieving the necessary levels of strength, notch toughness, and quality, the availability for review of welding filler metal documentation and welding procedure specifications (WPS) is required. This allows a thorough review on the part of the engineer and allows the engineer to have outside consultants review these documents, if needed.

(3) The fabricator and erector maintain written records of welding personnel qualification testing. Such records should contain information regarding date of testing, process, WPS, test plate, position, and the results of the testing. In order to verify the six-month limitation on welder qualification, the fabricator and erector should also maintain a record documenting the dates that each welder has used a particular welding process.

(4) The fabricator should consider AISC *Code of Standard Practice for Structural Stainless Steel Buildings* Section 6.1, in establishing material control procedures for structural stainless steel.

N4. INSPECTION AND NONDESTRUCTIVE TESTING PERSONNEL

1. Quality Control Inspector Qualifications

The fabricator or erector determines the qualifications, training, and experience required for personnel conducting the specified inspections. Qualifications should be based on the actual work to be performed and should be incorporated into the fabricator's or erector's QC program. Inspection of welding should be performed by an individual who, by training and/or experience in metals fabrication, inspection, and testing, is competent to perform inspection of the work. This is in compliance with AWS D1.6/D1.6M, clause 8 (AWS, 2017). Recognized certification programs are a method of demonstrating some qualifications but they are not the only method nor are they required by Chapter N for QC inspectors.

2. Quality Assurance Inspector Qualifications

The QA agency determines the qualifications, training, and experience required for personnel conducting the specified QA inspections. This may be based on the actual work to be performed on any particular project. AWS D1.6/D1.6M, clause 8.1.4.1, states, "An individual who, by training or experience, or both, in metals fabrication, inspection and testing, is competent to perform inspection of the work." Qualification for the QA inspector may include experience, knowledge, and physical requirements. These qualification requirements are documented in the QA agency's written practice. AWS B5.1 (AWS, 2013) is a resource for qualification of a welding inspector.

The use of assistant welding inspectors under direct supervision is as permitted in AWS D1.6/D1.6M, clause 8.1.4.4.

3. NDT Personnel Qualifications

NDT personnel should have sufficient education, training, and experience in those NDT methods they will perform. ASNT SNT-TC-1a (ASNT, 2011a) and ANSI/ASNT CP-189 (ASNT, 2011b) prescribe visual acuity testing, topical outlines for training, written knowledge, hands-on skills examinations, and experience levels for the NDT methods and levels of qualification.

As an example, under the provisions of ASNT SNT-TC-1a, an NDT Level II individual should be qualified to set up and calibrate equipment and to interpret and evaluate results with respect to applicable codes, standards, and specifications. The NDT Level II individual should be thoroughly familiar with the scope and limitations of the methods for which they are qualified and should exercise assigned responsibility for on-the-job training and guidance of trainees and NDT Level I personnel. The NDT Level II individual should be able to organize and report the results of NDT tests.

N5. MINIMUM REQUIREMENTS FOR INSPECTION OF STRUCTURAL STAINLESS STEEL BUILDINGS

1. Quality Control

The welding inspection tasks listed in Tables N5.4-1 through N5.4-3 are inspection items contained in AWS D1.6/D1.6M (AWS, 2017), but have been organized in the tables in a more rational manner for scheduling and implementation using categories of before welding, during welding, and after welding. Similarly, the bolting inspection tasks listed in Tables N5.6-1 through N5.6-3 are inspection items contained in the RCSC *Specification* (RCSC, 2014) but they have been organized in a similar manner for scheduling and implementation using traditional categories of before bolting, during bolting, and after bolting. The details of each table are discussed in Commentary Sections N5.4 and N5.6.

Typical model building codes, such as the 2018 *International Building Code* (IBC) (ICC, 2018), make specific statements about inspecting to "approved construction documents" the original and revised design drawings and specifications as approved by the building official or AHJ. AISC *Code of Standard Practice for Structural Stainless Steel Buildings* Section 4.2.1(a) (AISC, 2021) requires the transfer of information from the contract documents (design documents and project specifications) into accurate and complete fabrication and erection documents. Therefore, relevant items in the design documents and project specifications that must be followed in fabrication and erection should be placed on the fabrication and erection documents or in typical notes issued for the project. Because of this provision, QC inspection may be performed using fabrication and erection documents, not the original design documents.

The applicable referenced standards in construction documents are commonly this standard, the AISC *Code of Standard Practice for Structural Stainless Steel Buildings*, AWS D1.6/D1.6M, and the RCSC Specification.

2. Quality Assurance

AISC *Code of Standard Practice for Structural Stainless Steel Buildings* Section 8.4.2 contains the following provisions regarding the scheduling of shop fabrication inspection: "Inspection of shop work by the Inspector shall be performed in the Fabricator's shop to the fullest extent possible. Such inspections shall be timely, in-sequence, and performed in such a manner as will not disrupt fabrication operations and will permit the repair of nonconforming work prior to any required painting while the material is still in-process in the fabrication shop."

Similarly, AISC *Code of Standard Practice for Structural Stainless Steel Buildings* Section 8.4.3 states, "Inspection of field work shall be promptly completed without delaying the progress or correction of the work."

AISC *Code of Standard Practice for Structural Stainless Steel Buildings* Section 8.4.1 states, "The Fabricator and the Erector shall provide the Inspector with access to all places where the work is being performed. A minimum of 24 hours notification shall be given prior to the commencement of work." However, the inspector's timely inspections are necessary for this to be achieved, while the scaffolding, lifts, or other means provided by the fabricator or erector for their personnel are still in place or are readily available.

AISC *Code of Standard Practice for Structural Stainless Steel Buildings* Section 6.1 states, "Identification of Material. The fabricator shall be able to demonstrate by a written procedure and actual practice a method of material identification, visible up to the point of assembling members…"

AISC *Code of Standard Practice for Structural Stainless Steel Buildings* Section 8.2 states, "Material test reports shall constitute sufficient evidence that the mill product satisfies material order requirements. The Fabricator shall make a visual inspection of material that is received from the mill, ..." AISC *Code of Standard Practice for Structural Stainless Steel Buildings* Sections 5.2 and 6.1 address the traceability of material test reports to individual pieces of stainless steel and the identification requirements for structural stainless steel in the fabrication stage.

Model building codes, such as the IBC, make specific statements about inspecting to "approved construction documents" and the original and revised design documents and specifications as approved by the building official or the AHJ. Because of these IBC provisions, the QAI should inspect using the original and revised design documents and project specifications. The QAI may also use the fabrication and erection documents to assist in the inspection process.

3. Coordinated Inspection

Coordination of inspection tasks may be needed for fabricators in remote locations or distant from the project itself or for erectors with projects in locations where inspection by a local firm or individual may not be feasible or where tasks are redundant.

The approval of both the AHJ and EOR is required for quality assurance to rely upon quality control, so there must be a level of assurance provided by the quality activities that are accepted. It may also serve as an intermediate step short of waiving QA as described in Section N6.

4. Inspection of Welding

AWS D1.6/D1.6M requires inspection, and any inspection task should be done by the fabricator or erector (termed "contractor" within AWS D1.6/D1.6M) under the terms of clause 8.1.2.1, as follows:

> Contractor's Inspection. This type of inspection and test shall be performed as necessary prior to assembly, during assembly, during welding, and after welding to ensure that materials and workmanship meet the requirements of the contract documents. Contractor's inspection and testing shall be the responsibility of the Contractor, unless otherwise provided in the contract documents.

This is further clarified in clause 8.1.3.3, which states:

> Inspector. When the term Inspector is used without further qualification as to the specific Inspector category described above, it applies equally to inspection and verification within the limits of responsibility described in 8.1.2.1 and 8.1.2.2.

The basis of Tables N5.4-1, N5.4-2, and N5.4-3 are inspection tasks, as well as quality requirements, and related detailed items contained within AWS D1.6/D1.6M. Commentary Tables C-N5.4-1, C-N5.4-2, and C-N5.4-3 provide specific references to clauses in AWS D1.16/D1.6M. In the determination of the task lists, and whether the task is designated "observe" or "perform," the pertinent terms of the following AWS D1.6D1.6M clauses were used:

TABLE C-N5.4-1
Reference to AWS D1.6/D1.6M (AWS, 2017)
Clauses for Inspection Tasks prior to Welding

Inspection Tasks prior to Welding	Clauses
Welding procedure specifications (WPS) available	8.3
Manufacturer certifications for welding consumables available	8.2
Material identification (type/grade)	8.2
Welder identification system	8.4 (welder qualification) (identification system not required by AWS D1.6/D1.6M)
Fit-up of groove welds (including joint geometry) • Joint preparation • Dimensions (alignment, root opening, root face, bevel) • Cleanliness (condition of stainless steel surfaces) • Tacking (tack weld quality and location) • Backing type and fit (if applicable)	 8.5.2 7.8.2, 7.8.3 7.4 7.13 7.8.1.1, 7.8.1.2, 7.9
Fit-up of CJP groove welds of HSS T-, Y-, and K-joints without backing (including joint geometry) • Joint preparation • Dimensions (alignment, root opening, root face, bevel) • Cleanliness (condition of stainless steel surfaces) • Tacking (tack weld quality and location)	 7.8.2, 7.8.3 7.8.2, 7.8.3 7.4.3, 7.4.4 7.13
Configuration and finish of access holes	7.4.7, 8.5.2 (also see Section J1.6)
Fit-up of fillet welds • Dimensions (alignment, gaps at root) • Cleanliness (condition of stainless steel surfaces) • Tacking (tack weld quality and location)	 7.8.1 7.4 7.13

8.5 Inspection of Work and Records

8.5.1 Size, Type, Length, and Location of Welds. The Inspector shall ensure that the size, type, length, and location of all welds conform to the requirements of this code and to the detail drawings and that no unspecified welds have been added without the approval of the Engineer.

8.5.2 Scope of Inspection. The Inspector shall, at suitable intervals, observe joint preparation, assembly practice, welding techniques, and welder's and welding operator's performance, to ensure that the applicable requirements of this code are met.

TABLE C-N5.4-2
Reference to AWS D1.6/D1.6M (AWS, 2017)
Clauses for Inspection Tasks during Welding

Inspection Tasks during Welding	Clauses
Use of qualified welders	8.4
Control and handling of welding consumables • Packaging • Exposure control	8.2 7.3 7.3.1 (for SMAW), 7.3.2 (for SAW)
No welding over cracked tack welds	7.13
Environmental conditions • Wind speed within limits • Precipitation and temperature	 7.11 9.6.2.4
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)	7.8.4, 7.14, 8.3.2, 8.5.2 7.10
Welding techniques • Interpass and final cleaning • Each pass within profile limitations • Each pass meets quality requirements	7.15.2, 8.5.2, 8.5.3 7.20.1

> 8.5.3 Extent of Examination. The Inspector shall inspect the work to ensure that it meets the requirements of this code. ... Size and contour of welds shall be measured with suitable gages. ...

"Observe" tasks are as described in clauses 8.5.2 and 6.5.3. Clause 8.5.2 uses the term "observe" and also defines the frequency to be "at suitable intervals." "Perform" tasks are required for each weld by AWS D1.6/D1.6M, as stated in clause 8.5.1 or 8.5.3, or are necessary for final acceptance of the weld or item. The use of the term perform is based upon the use in AWS D1.6/D1.6M of the phrases "shall examine the work" and "size and contour of welds shall be measured"; hence, "perform" items are limited to those functions typically performed at the completion of each weld.

The words "all welds" in clause 8.5.1 clearly indicate that all welds are required to be inspected for size, length, and location in order to verify conformity. Chapter N follows the same principle in labeling these tasks "perform," which is defined as "Perform these tasks for each welded joint or member."

TABLE C-N5.4-3
Reference to AWS D1.6/D1.6M (AWS, 2017) Clauses for Inspection Tasks after Welding

Inspection Tasks after Welding	Clauses[a]
Welds cleaned	7.20.1
Size, length, and location of welds	8.5.1
Welds meet visual acceptance criteria • Crack prohibition • Weld/base-metal fusion • Crater cross section • Weld profiles • Weld size • Undercut • Porosity	8.5.3 Table 8.1(1) Table 8.1(2) Table 8.1(3) 7.16, Table 8.1(4) Table 8.1(6) Table 8.1(7) Table 8.1(8)
Arc strikes	7.19
k-area[a]	Not addressed in AWS
Weld access holes in rolled heavy shapes and built-up heavy shapes	7.4.7, 8.5.2 (see also Section J1.6)
Backing removed and weld tabs removed (if required)	7.9, 7.17.3, 8.17.2.1
Repair activities	7.21, 8.5.3
Document acceptance or rejection of welded joint or member	8.5.4, 8.5.5

[a] *k*-area issues were identified in AISC (1997). See Commentary on the 2016 AISC *Specification* Section J10.8.

The words "suitable intervals" used in clause 8.5.2 characterize that it is not necessary to inspect these tasks for each weld, but inspect as necessary to verify that the applicable requirements of AWS D1.1/D1.1M are met. Following the same principles and terminology, Chapter N labels these tasks as "observe," which is defined as "Observe these items on a random basis."

The selection of suitable intervals as used in AWS D1.6/D1.6M is not defined within AWS D1.6/D1.6M, other than the AWS statement "to ensure that the applicable requirements of this code are met." The establishment of "at suitable intervals" is dependent upon the quality control program of the fabricator or erector, the skills and knowledge of the welders themselves, the type of weld, and the importance of the weld. During the initial stages of a project, it may be advisable to have increased levels of observation to establish the effectiveness of the fabricator's or erector's quality control program, but such increased levels need not be maintained for the duration of the project, nor to the extent of inspectors being on site. Rather, an appropriate level of observation intervals can be used that are commensurate with the observed performance of the contractor and their personnel. More inspection may be warranted for weld fit-up and monitoring of welding operations for complete-joint-penetration (CJP) and partial-joint-penetration (PJP) groove welds loaded in transverse tension, compared to the time spent on groove welds loaded in compression or shear, or time spent on fillet welds. More time may be warranted observing welding operations for multi-pass fillet welds, where poor quality root passes and poor fit-up may be obscured by subsequent weld beads, when compared to single pass fillet welds.

The terms "perform" and "observe" are not to be confused with the terms "periodic special inspection" and "continuous special inspection" used in the IBC for other construction materials. Both sets of terms establish two levels of inspection. The IBC terms specify whether the inspector is present at all times or not during the course of the work. Chapter N establishes inspection levels for specific tasks within each major inspection area. "Perform" indicates each item is to be inspected, and "observe" indicates samples of the work are to be inspected. It is likely that the number of inspection tasks will determine whether an inspector has to be present full time, but it is not in accordance with Chapter N to let the time an inspector is on site determine how many inspection tasks are done.

AWS D1.6/D1.6M, clause 8.3, states that the contractor's (fabricator/erector) inspector is specifically responsible for the WPS, verification of prequalification or proper qualification, and performance in compliance with the WPS. Quality assurance inspectors monitor welding to make sure QC is effective. For this reason, Tables N5.4-1 and N5.4-2 maintain an inspection task for the QA for these functions. For welding to be performed, and for this inspection work to be done, the WPS must be available to both welder and inspector. A separate inspection for HSS T-, Y-, and K-connections was added to recognize the separate fit-up tolerances for these joints in AWS D1.6/D1.6M, Table 8.1, and their importance to achieving an acceptable root.

Material verification of weld filler materials is accomplished by observing that the consumable markings correspond to those in the WPS and that certificates of compliance are available for consumables used.

The footnote to Table N5.4-1 states that "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 EOR and shall be the low-stress type and compatible with the stainless steel base metal." AWS D1.6/D1.6M does not require a welding personnel identification system. However, the inspector must verify the qualifications of welders, including identifying those welders whose work "appears to be below the requirements of this code." Also, if welds are to receive nondestructive testing (NDT), it is essential to have a welding personnel identification system to reduce the rate of NDT for good welders and increase the rate of NDT for welders whose welds frequently fail NDT. This welder identification system can also benefit the contractor by clearly identifying welders who may need additional training.

Table N5.4-3 includes requirements for observation that "No prohibited welds have been added without the approval of the EOR." AWS D1.6/D1.6M, clause 7.13, includes specific provisions for tack welds incorporated into final welds, tack welds not incorporated into final welds, and construction aid welds.

AWS D1.6/D1.6M, clause 9, Stud Welding, includes requirements regarding the stud welding materials and their condition, base metal condition, stud application qualification testing, pre-production welding inspection and bend testing, qualification of the welding operator, visual inspection of completed studs and bend testing of certain studs when required, and the repair of nonconforming studs. For manually welded studs, special requirements apply to the stud base and the welding procedures.

The proper fit-up for groove welds and fillet welds prior to welding should first be checked by the fitter and/or welder. Such detailed dimensions should be provided on the shop or erection drawings, as well as included in the WPS. Fitters and welders must be equipped with the necessary measurement tools to verify proper fit-up prior to welding.

AWS D1.6/D1.6M, clause 8.2, Inspection of Materials, states that, "The Contractor's Inspector shall ensure that only materials conforming to the requirements of the contract documents and this code are used." For this reason, the check of welding equipment is assigned to QC only in Chapter N, and is not required for QA.

5. Nondestructive Testing of Welded Joints

5a. Procedures

Buildings are subjected to static loading unless fatigue is specifically addressed as prescribed in Appendix 3. Section J2 provisions contain exceptions to AWS D1.6/D1.6M.

5b. CJP Groove Weld NDT

For statically loaded structures, AWS D1.6/D1.6M and the Specification have no specific nondestructive testing (NDT) requirements, leaving it to the engineer to determine the appropriate NDT method(s), locations or categories of welds to be tested, and the frequency and type of testing (full, partial, or spot), in accordance with AWS D1.6/D1.6M, clause 8.15.

The Specification implements a selection of NDT methods and a rate of ultrasonic testing (UT) based upon a rational system of risk of failure. If based upon a model building code, such as the *International Building Code* (ICC, 2018), the applicable building code will assign every building or structure to one of four different risk categories. Where there is no applicable building code, then Section A1 requires that the risk category be assigned in accordance with ASCE/SEI 7 (ASCE, 2016).

Complete-joint-penetration (CJP) groove welds loaded in tension applied transversely to their axis are assumed to develop the capacity of the smaller stainless steel element being joined and therefore have the highest demand for quality. CJP groove welds in compression or shear are not subjected to the same crack propagation risks as welds subjected to tension. Partial-joint-penetration (PJP) groove welds are designed using a limited available strength when in tension, based upon the root condition, and therefore are not subjected to the same high stresses and subsequent crack propagation risk as a CJP groove weld. PJP groove welds in compression or shear are similarly at substantially less risk of crack propagation than CJP groove welds.

Fillet welds are designed using limited strengths, similar to PJP groove welds, and are designed for shear stresses regardless of load application and therefore do not warrant NDT.

The selection of joint type for ultrasonic testing (UT) is based upon AWS D1.6/D1.6M, clause 8.20, which limits the procedures and standards as stated in Part F of AWS D1.6D1.6M to groove welds and heat-affected zones in austenitic stainless steel. The requirement to inspect 10% of CJP groove welds is a requirement that the full length of 10% of the CJP groove welds are to be inspected.

5c. Welded Joints Subject to Fatigue

CJP groove welds in butt joints so designated in Appendix 3, Table A-3.1, Sections 5 and 6.1, require that internal soundness be verified using ultrasonic testing (UT) or radiographic testing (RT), meeting the acceptance requirements of AWS D1.6/D1.6M, clause 8.12 or 8.13, as appropriate.

5e. Reduction of Ultrasonic Testing Rate

For statically loaded structures in risk categories III and IV, reduction of the rate of UT from 100% is permitted for individual welders who have demonstrated a high level of skill, proven after a significant number of their welds have been tested.

5f. Increase in Ultrasonic Testing Rate

For risk category II, where 10% of CJP groove welds loaded in transverse tension are tested, an increase in the rate of UT is required for individual welders who have failed to demonstrate a high level of skill, established as a failure rate of more than 5%, after a sufficient number of their welds have been tested. To implement this effectively, and not necessitate the retesting of welds previously deposited by a welder who has a high reject rate established after the 20 welds have been tested, it is suggested that at the start of the work, a higher rate of UT be performed on each welder's completed welds.

6. Inspection of Bolting

The RCSC *Specification* (RCSC, 2014), like the referenced welding standard, defines bolting inspection requirements in terms of inspection tasks and scope of examinations. The RCSC *Specification* uses the term "routine observation" for the inspection of all pretensioned bolts, further validating the choice of the term "observe" in this chapter of the Specification.

Table N5.6-1 includes requirements for observation of "Fasteners marked in accordance with ASTM requirements." This includes the required package marking of the fasteners and the product marking of the fastener components in accordance with the applicable ASTM standard.

Snug-tightened joints are required to be inspected to verify that the proper fastener components are used and that the faying surfaces are brought into firm contact during installation of the bolts. The magnitude of the clamping force that exists in a snug-tightened joint is not a consideration and need not be verified.

Pretensioned joints and slip-critical joints are required to be inspected to verify that the proper fastener components are used and that the faying surfaces are brought into firm contact during the initial installation of the bolts. Pre-installation verification testing is required for all pretensioned bolt installations, and the nature and scope of installation verification will vary based on the installation method used. The following provisions from the RCSC *Specification* serve as the basis for Tables N5.6-1, N5.6-2, and N5.6-3. In the following, underlining has been added for emphasis of terms:

> **9.2.1. Turn-of-Nut Pretensioning:** The inspector shall observe the pre-installation verification testing required in Section 8.2.1. Subsequently, it shall be ensured by <u>routine observation</u> that the bolting crew properly rotates the turned element relative to the unturned element by the amount specified in Table 8.2. Alternatively, when fastener assemblies are match-marked after the initial fit-up of the joint, but prior to pretensioning; visual inspection after pretensioning is permitted in lieu of routine observation.
>
> **9.2.2. Calibrated Wrench Pretensioning:** The inspector shall <u>observe</u> the pre-installation verification testing required in Section 8.2.2. Subsequently, it shall be ensured by <u>routine observation</u> that the bolting crew properly applies the calibrated wrench to the turned element. No further evidence of conformity is required.

The presence of the inspector is dependent upon whether the installation method provides visual evidence of completed installation. Turn-of-nut installation without matchmarking and calibrated wrench installation provides no visual evidence of completed installation, and the inspector is to be “engaged” on site, although not necessarily watching every bolt or joint as it is being pretensioned.

The inspection provisions of the RCSC Specification rely upon observation of the work; hence, all tables in this Specification use “observe” for the designated tasks.

7. Other Inspection Tasks

IBC requires that anchor rods be set accurately to the pattern and dimensions called for on the plans. In addition, it is required that the protrusion of the threaded ends through the connected material be sufficient to fully engage the threads of the nuts, but not be greater than the length of the threads on the bolts.

AISC *Code of Standard Practice for Structural Stainless Steel Buildings* Section 7.5.1 states that anchor rods, foundation bolts, and other embedded items are to be set by the owner’s designated representative for construction. The erector is likely not on site to verify placement, therefore it is assigned solely to the quality assurance inspector (QAI). Because it is not possible to verify proper anchor rod materials and embedment following installation, it is required that the QAI be on site when the anchor rods are being set.

N6. APPROVED FABRICATORS AND ERECTORS

IBC Section 1704.2.5.1 (ICC, 2018) states that:

> Special inspections during fabrication are not required where the work is done on the premises of a fabricator registered and approved to perform such work without special inspection.
>
> Approval shall be based upon review of the fabricator's written procedural and quality control manuals and periodic auditing of fabrication practices by an approved agency.

An example of how these approvals may be made by the building official or authority having jurisdiction (AHJ) is the use of the AISC Certification program. A fabricator certified to the AISC Certified Structural Steel Fabricator, *Standard for Certification Programs* (AISC, 2020), meets the criteria of having a quality control manual, written procedures, and annual onsite audits conducted by AISC's independent auditing company, Quality Management Company, LLC. Similarly, steel erectors may be an AISC Certified Erector or AISC Advanced Certified Steel Erector. The audits confirm that the company has the personnel, knowledge, organization, equipment, experience, capability, procedures, and commitment to produce the required quality of work for a given certification category.

Granting a waiver of QA inspections in a fabrication shop does not eliminate the required NDT of welds; instead of being performed by QA, such inspections are performed by the fabricator's QC. Even when QA inspection is waived, the NDT reports prepared by the fabricator's QC are available for review by a third-party QA.

APPENDIX 1

DESIGN BY ADVANCED ANALYSIS

General provisions for designing for stability are presented in Chapter C, in which specific details are provided for the direct analysis method. This appendix provides details for explicitly modeling system and member imperfections and/or inelasticity within the analysis. The provisions are based on the 2016 AISC *Specification*, but with the modifications described herein. Further information can be found in the Commentary to the 2016 AISC *Specification*.

1.2. DESIGN BY ELASTIC ANALYSIS

1. General Stability Requirements

This method is currently restricted to doubly symmetric sections, including I-shapes, HSS, and box sections, because current analytical testing has generally taken place with these section types. The designer can consider using singly symmetric shapes or other shapes as long as an investigation is undertaken to verify the results are properly capturing twisting effects and generally produce designs comparable to the traditional design approach as specified in Chapter C and the other design requirements contained in Chapters D through K.

2. Calculation of Required Strengths

Since member instability is directly captured in a second-order analysis with imperfections, only cross-section strength checks are required to verify the capacity of the structure.

Note that in the 2016 AISC *Specification*, the use of additional notional loads in place of stiffness reduction through τ_b is not permitted; the same restriction is applied in this Specification for stainless steel design.

1.3. DESIGN BY INELASTIC ANALYSIS

This section contains provisions for the inelastic analysis and design of structural stainless steel systems by second-order inelastic analysis, including continuous beams, moment frames, braced frames, and combined systems. The nonlinear material stress-strain response of stainless steel has a direct influence on the structural behavior of stainless steel cross sections, members, and frames (Gardner, 2019). Material nonlinearity results in greater deflections due to the loss of material stiffness, and therefore, without the consideration of plasticity in the structural analysis, forces and moments are often underestimated (Walport et al., 2019a). It has been concluded that material nonlinearity should generally be considered in the global analysis of stainless steel structures (Walport et al., 2019b). The guidance provided in the 2016 AISC *Specification* for carbon steel design largely relates to the occurrence of traditional idealized plastic hinges. However, such hinges do not form in stainless steel structures; instead, zones of plasticity with gradually reducing stiffness are exhibited. Traditional plastic hinge analysis is, therefore, not generally appropriate, and an inelastic plastic zone (distributed-plasticity) analysis, incorporating a nonlinear material stress-strain response, must be carried out for stainless steel design, as given in this appendix.

2a. Material

Extensive past research on the plastic and inelastic behavior of continuous beams, rigid frames, and connections has amply demonstrated the suitability of steel with yield stress levels up to 80 ksi (550 MPa) (Afshan et al., 2015; Gardner et al., 2006).

2b. Unbraced Length

The equations for calculating the limiting unbraced length, L_{pd}, are a modified version of those in the 2016 AISC *Specification* for carbon steel reflecting the changes in Chapter F for the two materials. The limiting unbraced length, L_{pd}, represents a portion of the limiting laterally unbraced length for the limit state of yielding, L_p, defined in Chapter F. To calibrate the revised expressions, values of E = 29,000 ksi (200 000 MPa), F_y = 41 ksi (280 MPa), and two ratios, 1 and 0.8, were considered. With these values and using the expressions from the 2016 AISC *Specification*, when $M_1'/M_2 = 1$ and when $M_1'/M_2 = 0.8$, the limiting unbraced length is equal to 0.67 and 0.90 times the limiting laterally unbraced length for the limit state of yielding, respectively. Consequently, forming simultaneous equations and using the revised definition of the limiting laterally unbraced length for the limit state of yielding defined in Chapter F for stainless steel, the two constants were derived in each equation for L_{pd}.

2c. Axial Force

For consistency, the limitation on the design compressive strength of $0.75F_yA_g$ is from the 2016 AISC *Specification*. This precautionary limitation has been included to provide sufficient inelastic rotation capacity in members subject to high levels of axial force. Note, however, that the continuous strength method presented in Appendix 2 results in strain limits that inherently capture the influence of the loading on ductility.

3. Analysis Requirements

The second-order inelastic structural analysis is carried out using finite element analysis utilizing beam elements. With the nonlinear stainless steel material model given in Appendix 7, capacities continuously increase under increasing deformation. In reality, cross-section deformations and capacities are controlled by local buckling. A limit is therefore required to control the level of plastic deformation and hence the capacity of the cross section and ultimately the structure. The limit comes from the continuous strength method (CSM). This approach of design by inelastic analysis therefore features a second-order inelastic analysis using the nonlinear stainless steel material model, included in Appendix 7, and the CSM strain limits. Hence, cross-section slenderness-dependent levels of spread of plasticity, moment redistribution, and strain hardening are permitted, in a consistent and accurate manner.

When performing this type of analysis, failure of the structural system or member is defined as the point at which either (1) the CSM strain limit is reached at any point in the system or (2) the peak load is attained during the analysis, whichever occurs first (Walport et al., 2019c). The maximum load obtained from the analysis is a nominal load. Therefore, in order to determine the design strength of the structural system or member, the nominal load has to be converted into a design load by multiplying it by the resistance factor, $\phi = 0.90$.

The continuous nature of the approach allows cross sections of all classes to be designed in the same way.

3c. Residual Stress and Partial Yielding Effects

As an alternative to modeling residual stresses, equivalent member imperfections may be utilized that account for both geometric imperfections and material imperfections (i.e., residual stresses), as determined using Equation C-A-1-1, where α_{eq} is the imperfection factor as given in Table C-A-1.1 (Walport et al., 2020). These may be modeled in the form of a half sine wave or elastic buckling mode shape:

$$\frac{e_0}{L} = \frac{\alpha_{eq}}{150} \text{ but } \frac{e_0}{L} \geq \frac{1}{1000} \tag{C-A-1-1}$$

where

L = length of member, in. (mm)

e_0 = equivalent member imperfection amplitude, in. (mm)

3d. Continuous Strength Method Strain Limits

The most accurate representation of the behavior of a structure is achieved through directly allowing for instability, plasticity, residual stresses, and initial geometric imperfections through the use of second-order (advanced) inelastic analysis with imperfections. Advanced analysis is commonly carried out using beam finite elements for ease of use and computational efficiency, but these elements cannot capture local cross-section deformations. While shell finite elements are able to capture cross-section local buckling behavior, they are computationally expensive. A practical solution is to use beam finite elements, with strain limits applied to simulate local buckling and control the level of plastic deformation, and hence the capacity of the cross section and ultimately the structure. In this approach (Fieber et al., 2019, 2020; Walport et al., 2021), the influence of material nonlinearity on the structural response is directly modeled through the definition of the full stress-strain curve of the material, while the strain limits are taken from the continuous strength method (CSM).

TABLE C-A-1.1
Imperfection Factors, α_{eq}, for Different Types of Members for Calculating the Equivalent Member Imperfection

Member Type	Axis of Buckling	α_{eq}
Rectangular and round HSS	Any	0.49
Rolled or welded I-shaped sections, and welded box sections	Major	0.49
	Minor	0.76

The accuracy of this design approach was verified against a significant number of benchmark results from experiments and nonlinear shell finite element analyses of stainless steel members and structural systems. The continuous strength method (CSM) is employed to simulate local buckling by applying strain limits to control the extent to which plasticity, moment redistribution, and strain hardening is permitted. Failure of a member or structure is defined as the point at which the CSM strain limit is reached or, in stability governed cases, where the peak load occurs prior to the CSM strain limit being reached; the peak load is taken as the ultimate load. Equations A-1-5 and A-1-6 differ from Equations A-2-3 and A-2-4 to allow for gradual yielding due to the incorporation of the nonlinear material model.

The required strain is determined at the outer compressive fiber and may be averaged over the elastic local buckling half-wavelength, L_{el}, of the full cross section. Note that the presence of stiffeners locally constrains the shape of the cross section and hence L_{el} is located to either side of the stiffener, as shown in Figure C-A-1.1.

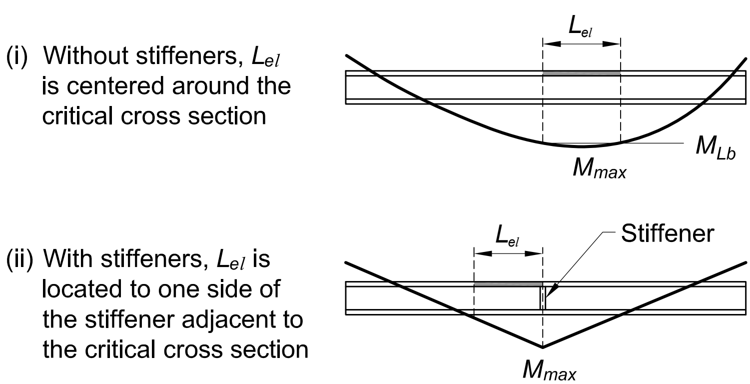


Fig. C-A-1.1. Moment diagrams and strain averaging approach.

The local cross-section slenderness, λ_l, quantifies the susceptibility of a cross section to local buckling. The elastic buckling stress of the crosssection, F_{el}, may be calculated accurately through numerical methods, for example, the Constrained and Unconstrained Finite Strip Method (CUFSM) software (Li and Schafer, 2010) or, for I-shaped sections, square and rectangular HSS, and box sections, using the simplified expressions presented in the following discussion (Gardner et al., 2019).

Second-order inelastic analysis with CSM strain limits enables the resistance of the structure to be captured allowing suitable levels of inelastic redistribution within indeterminate structural systems; it also allows the structural failure mechanism to be clearly visualized (Fieber et al., 2020; Walport et al., 2021).

Elastic Local Buckling Stress, F_{el}, of Full Cross Section. The expressions for the determination of F_{el} for full cross sections provided in this Commentary are for I-sections, square and rectangular HSS, and box sections. Expressions for other cross-section types can be found in Gardner et al. (2019). Design examples illustrating the use of these expressions can be found in AISC Design Guide 27 (Baddoo and Meza, 2021).

The full cross-section elastic local buckling stress, F_{el}, is determined using Equation C-A-1-2:

$$F_{el} = F^{SS}_{el,p} + \zeta\left(F^{F}_{el,p} - F^{SS}_{el,p}\right) \text{ where } 0 \le \zeta \le 1 \quad \text{(C-A-1-2)}$$

where

$F^{SS}_{el,p}$ = elastic local buckling stress of isolated critical element assuming simply supported boundary conditions, ksi (MPa)

$$= \min\left(\beta_f F^{SS}_{el,f}, \beta_w F^{SS}_{el,w}\right) \quad \text{(C-A-1-3)}$$

$F^{SS}_{el,f}$ = elastic local buckling stress of isolated flange assuming simply supported boundary conditions, ksi (MPa)

$F^{SS}_{el,w}$ = elastic local buckling stress of isolated web assuming simply supported boundary conditions, ksi (MPa)

$F^{F}_{el,p}$ = elastic local buckling stress of isolated critical element assuming fixed boundary conditions, ksi (MPa)

$$= \min\left(\beta_f F^{F}_{el,f}, \beta_w F^{F}_{el,w}\right) \quad \text{(C-A-1-4)}$$

$F^{F}_{el,w}$ = elastic local buckling stress of isolated web assuming fixed boundary conditions, ksi (MPa)

$F^{F}_{el,f}$ = elastic local buckling stress of isolated flange assuming fixed boundary conditions, ksi (MPa)

β_f = flange stress correction factor

$$= \frac{F_{cs,max}}{F_{f,max}} \quad \text{(C-A-1-5)}$$

β_w = web stress correction factor

$$= \frac{F_{cs,max}}{F_{w,max}} \quad \text{(C-A-1-6)}$$

$F_{cs,max}$ = maximum compressive stress in the cross section, ksi (MPa)

$F_{f,max}$ = maximum compressive stress in the flange, ksi (MPa)

$F_{w,max}$ = maximum compressive stress in the web, ksi (MPa)

ζ = interaction coefficient as determined from Table C-A-1.4

Note: β_f and β_w are only necessary when the maximum compressive stress in the flange and web is different (e.g., I-sections subjected to minor-axis bending).

Elastic Local Buckling Stress of Isolated Web and Flange. The isolated flange and web elastic buckling stresses with simply supported (*SS*) and fixed (*F*) boundary conditions may be determined from Equation C-A-1-7:

$$F_{el,p} = k\frac{\pi^2 E}{12\left(1-\nu^2\right)}\left(\frac{t_p}{b_p}\right)^2 \tag{C-A-1-7}$$

where

b_p = width of element, in. (mm)

k = buckling coefficient based on the stress distribution, Ψ, as calculated from Tables C-A-1.2 and C-A-1.3

t_p = thickness of element, in. (mm)

ν = Poisson's ratio
= 0.3

Elastic Local Buckling Half-Wavelength, L_{el}, of Full Cross Section. A design example illustrating the use of these expressions can be found in AISC Design Guide 27.

The full cross-section elastic local buckling half-wavelength, L_{el}, is determined using Equation C-A-1-8:

$$L_{el} = L^{SS}_{el,p} + \zeta\left(L^{F}_{el,p} - L^{SS}_{el,p}\right) \text{ where } 0 \le \zeta \le 1 \tag{C-A-1-8}$$

where

ζ = interaction coefficient as determined from Table C-A-1.4

$L^{SS}_{el,p}$ = elastic local buckling half-wavelength of isolated critical element assuming simply supported boundary conditions, in. (mm)

$$= L^{SS}_{el,w}\eta + L^{SS}_{el,f}\left(1-\eta\right) \tag{C-A-1-9}$$

$L^{SS}_{el,w}$ = elastic local buckling half-wavelength of isolated web assuming simply supported boundary conditions, in. (mm)

$L^{SS}_{el,f}$ = elastic local buckling half-wavelength of isolated flange assuming simply supported boundary conditions, in. (mm)

η = transition function given in Table C-A-1.5

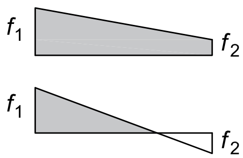
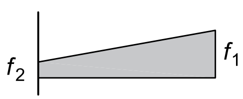
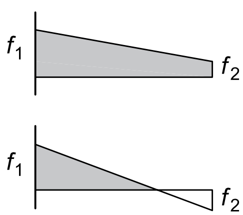
$L^{F}_{el,p}$ = elastic local buckling half-wavelength of isolated critical element assuming fixed boundary conditions, in. (mm)

$$= L^{F}_{el,w}\eta + L^{F}_{el,f}\left(1-\eta\right) \tag{C-A-1-10}$$

$L^{F}_{el,w}$ = elastic local buckling half-wavelength of isolated web assuming fixed boundary conditions, in. (mm)

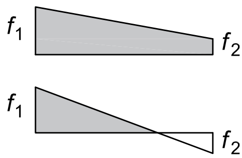
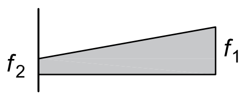
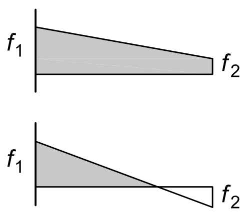
$L^{F}_{el,f}$ = elastic local buckling half-wavelength of isolated flange assuming fixed boundary conditions, in. (mm)

TABLE C-A-1.2
Buckling Coefficient, *k*, for Stiffened and Unstiffened Elements with Simply Supported (SS) Boundary Conditions along Adjoined Edges

Stress Distribution[a]		$\psi = \sigma_2/\sigma_1$	Buckling Coefficient
Stiffened Elements (SS – SS)		1	$k = 4$
		$1 > \psi > 0$	$k = \dfrac{8.2}{1.05 + \psi}$
		0	$k = 7.81$
		$0 > \psi > -1$	$k = 9.78\psi^2 - 6.29\psi + 7.81$
		–1	$k = 23.9$
Unstiffened Elements (SS – free)		1	$k = 0.43$
		0	$k = 0.57$
		–1	$k = 0.85$
		$1 > \psi \geq -3$	$k = 0.07\psi^2 - 0.21\psi + 0.57$
		1	$k = 0.43$
		$1 > \psi > 0$	$k = \dfrac{0.578}{0.34 + \psi}$
		0	$k = 1.70$
		$0 > \psi > -1$	$k = 17.1\psi^2 - 5\psi + 1.7$
		–1	$k = 23.8$
		$-1 > \psi \geq -3$	$k = 5.78\psi^2 - 12.8\psi + 5.22$

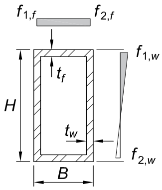
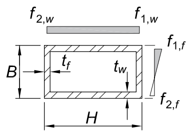
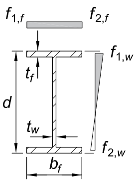
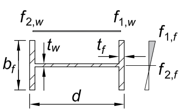
[a] Compression taken as positive (shaded gray).

TABLE C-A-1.3
Buckling Coefficient, *k*, or Stiffened and Unstiffened Elements with Fixed (F) Boundary Conditions along Adjoined Edges

Stress Distribution[a]		$\psi = \sigma_2/\sigma_1$	Buckling Coefficient
Stiffened Elements (F – F)		1	$k = 6.97$
		$1 > \psi > 0$	$k = \dfrac{14.29}{1.05 + \psi}$
		0	$k = 13.6$
		$0 > \psi > -1$	$k = 14.5\psi^2 - 11.5\psi + 13.6$
		–1	$k = 39.6$
Unstiffened Elements (F – free)		1	$k = 1.25$
		0	$k = 1.62$
		–1	$k = 2.13$
		$1 > \psi > -1$	$k = 0.07\psi^2 - 0.44\psi + 1.62$
		1	$k = 1.25$
		$1 > \psi > 0$	$k = \dfrac{1.58}{0.263 + \psi}$
		0	$k = 6.01$
		–1	$k = 37.3$
		$-1 > \psi \geq -3$	$k = 9.8\psi^2 - 21.5\psi + 6$

[a] Compression taken as positive (shaded gray).

TABLE C-A-1.4
Interaction Coefficient, ζ, for Square and Rectangular HSS, Box Sections, and I-Sections

Section		Load Case[a]	Flange Critical ($\phi < 1$)	Web Critical ($\phi \geq 1$)
Square and Rectangular HSS, and Box Sections		Compression and/or major-axis bending	$\zeta = \frac{t_w}{t_f}(0.24 - a_f\phi)^{0.6}$ $a_f = 0.24 - \left[0.1\left(\frac{t_f}{t_w}\right)^2\left(\frac{H}{B} - 1\right)\right]^{1/0.6} \leq 0.24$	$\zeta = \frac{t_f}{t_w}\left(0.53 - \frac{a_w}{\phi}\right)$ $a_w = 0.63 - 0.1\frac{H}{B} \leq 0.53$
		Compression and/or minor-axis bending	N/A	$\zeta = \frac{t_f}{t_w}\left(0.53 - \frac{a_w}{\phi}\right)$ $a_w = 0.63 - 0.1\frac{H}{B} \leq 0.53$
I-Sections		Compression and/or major-axis bending	$\zeta = 0.15\frac{t_f}{t_w}\phi \geq \frac{t_w}{t_f}(0.4 - 0.25\phi)$	$\zeta = \frac{t_f}{t_w}\left(0.45 - \frac{0.3}{\phi^2}\right)$
		Compression and/or minor-axis bending	$\zeta = a_f\frac{t_f}{t_w}\phi \geq \frac{t_w}{t_f}(a_{f1} - 0.25\phi)$ $a_f = 0.9 - 0.75\psi_f \leq 0.35$ $a_{f1} = 1 - 0.6\psi_f \leq 0.7$	$\zeta = \frac{t_f}{t_w}\left(0.45 - \frac{a_w}{\phi^2}\right)$ $a_w = 0.8\psi_f - 0.5 \geq 0.1$

$$\phi = \frac{\beta_f F_{el,f}^{SS}}{\beta_w F_{el,w}^{SS}} = \left(\frac{F_{el,f}^{SS}}{F_{el,w}^{SS}}\right)\left(\frac{F_{w,\max}}{F_{f,\max}}\right)$$

$$\psi_f = \frac{\sigma_{2,f}}{\sigma_{1,f}}$$

[a] For tension and bending, the expression for bending can conservatively be used.

TABLE C-A-1.5
Transition Function, η

Section	Load Case[a]	
	Compression and/or Major-Axis Bending	Compression and/or Minor-Axis Bending
Square and rectangular HSS and box sections	$\eta = 1 - \dfrac{1}{\phi^3 + 1}$	
I-sections	$\eta = 1 - \dfrac{1}{(\phi - 0.5)^3 + 1} \geq 0$	$\eta = 1 - \dfrac{1}{(\phi - 0.5a_1)^3 + 1} \geq 0$ Where $a_1 = 2\psi_f - 1 \geq -0.6$
$\phi = \dfrac{\beta_f F_{el,f}^{SS}}{\beta_w F_{el,w}^{SS}} = \left(\dfrac{F_{el,f}^{SS}}{F_{el,w}^{SS}}\right)\left(\dfrac{F_{w,\max}}{F_{f,\max}}\right)$ $\psi_f = \dfrac{\sigma_{2,f}}{\sigma_{1,f}}$ (see Table C-A-1.4)		
[a] For tension + bending, the expression for bending can conservatively be used.		

Elastic Local Buckling Half-Wavelength of Isolated Web and Flange. The isolated flange and web local buckling half-wavelength assuming simply supported (SS) and fixed (F) boundary conditions may be determined from Equation C-A-1-11:

$$L_{el,p} = k_{Lb} b_p \qquad \text{(C-A-1-11)}$$

where

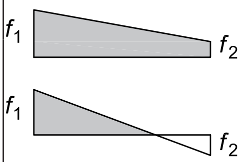
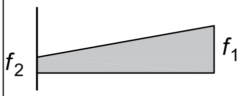
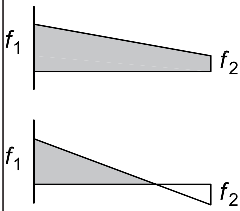
b_p = width of element, in. (mm)

k_{Lb} = half-wavelength coefficient given in Table C-A-1.6

3e. Shear, Torsion, and Bending Interaction

In this method of design, by second-order inelastic analysis with CSM strain limits, the influence of shear force on flexural strength is accounted for through the reduction factor, ρ_{csm}. Extensive numerical modeling has shown this to account for the shear and bending interaction and provide safe and accurate strength predictions (Walport et al., 2021).

TABLE C-A-1.6
Plate Buckling Half-Wavelength Coefficient, k_{Lb}, for Stiffened and Unstiffened Elements with Simply Supported and Fixed-Edge Conditions

<table>
<tr><th colspan="2">Stress Distribution[a]</th><th>$\psi = \sigma_2/\sigma_1$</th><th>Simply Supported Edges</th><th>Fixed Edges</th></tr>
<tr><td rowspan="3">Stiffened Elements</td><td rowspan="3"></td><td>$1 \geq \psi \geq 0.25$</td><td>$k_{Lb} = 1$</td><td>$k_{Lb} = 0.66$</td></tr>
<tr><td>$0.25 \geq \psi \geq -1$</td><td>$k_{Lb} = 1 - 0.21(\psi - 0.25)^2$</td><td>$k_{Lb} = 0.66 - 0.12(\psi - 0.25)^2$</td></tr>
<tr><td>-1</td><td>$k_{Lb} = 0.67$</td><td>$k_{Lb} = 0.47$</td></tr>
<tr><td rowspan="5">Unstiffened Elements</td><td></td><td>$1 \geq \psi \geq -1$</td><td>$k_{Lb} = \max\begin{cases} 2.8 + 0.3a_2\left(\dfrac{2d}{b_f}\right)\left(\dfrac{t_f}{t_w}\right) \\ 2 + 3a_2\phi \end{cases}$
For compression and/or major-axis bending[b]:
$a_2 = 1.0$
For compression and/or minor-axis bending[b]:
$a_2 = 2\psi \leq 2.6 - 1.6\psi$</td><td>$k_{Lb} = 1.65$</td></tr>
<tr><td rowspan="4"></td><td>$1 \geq \psi \geq 0$</td><td>$k_{Lb} = \max\begin{cases} 2.8 + 0.3a_2\left(\dfrac{2d}{b_f}\right)\left(\dfrac{t_f}{t_w}\right) \\ 2 + 3a_2\phi \end{cases}$
For compression and/or major-axis bending:
$a_2 = 1.0$
For compression and/or minor-axis bending:
$a_2 = 2\psi = 2.6 - 1.6\psi$</td><td>$k_{Lb} = 1.65$</td></tr>
<tr><td>$0 > \psi \geq -1$</td><td>$k_{Lb} = \dfrac{0.818}{0.221 - \psi}$</td><td>$k_{Lb} = \dfrac{0.68}{0.41 - \psi}$</td></tr>
<tr><td>-1</td><td>$k_{Lb} = 0.67$</td><td>$k_{Lb} = 0.48$</td></tr>
<tr><td>$-1 \geq \psi \geq -3$</td><td>$k_{Lb} = 0.06\psi^2 + 0.39\psi + 1$</td><td>$k_{Lb} = 0.04\psi^2 + 0.29\psi + 0.73$</td></tr>
<tr><td colspan="5">$\phi = \dfrac{\beta_f F_{el,f}^{SS}}{\beta_w F_{el,w}^{SS}} = \left(\dfrac{F_{el,f}^{SS}}{F_{el,w}^{SS}}\right)\left(\dfrac{F_{w,\max}}{F_{f,\max}}\right)$</td></tr>
<tr><td colspan="5">[a] Compression taken as positive (shaded gray).
[b] For tension plus bending, the expression for bending can conservatively be used.</td></tr>
</table>

APPENDIX 2
THE CONTINUOUS STRENGTH METHOD

The continuous strength method (CSM) is a deformation-based design approach which considers the beneficial effects of strain hardening and element interaction in the design of stainless steel cross sections. The CSM has been developed over a number of years (Ashraf et al., 2006; Ashraf et al., 2008; Afshan and Gardner, 2013; Liew and Gardner, 2015; Buchanan et al., 2016; Zhao and Gardner, 2018; Zhao et al., 2017). It uses the nondimensional measure of cross-section deformation capacity, which is presented as a function of the full cross-section slenderness that accounts for the beneficial effect of element interaction within the cross section. An elastic, linear hardening material model is also adopted, which gives a better representation of the actual material behavior of stainless steels than the elastic, perfectly plastic material model.

2.1. LIMITATIONS

The CSM applies to round HSS and sections comprising flat plates (e.g., doubly symmetric I-shapes, square and rectangular HSS, singly symmetric channel and T-sections, and asymmetric angle sections) subjected to both isolated and combined loading conditions.

Table A-2.1.1 gives the range of applicability of the CSM; the CSM is not applicable in the case of global instabilities.

2.2. MATERIAL MODELING

The CSM elastic, linear hardening material model is illustrated in Figure A-2.2.1, with the strain hardening slope, E_{sh}, calculated using Equation A-2-1. The values of the material coefficients for austenitic and duplex stainless steel were calibrated based on measured material tensile coupon test data by means of least squares regression (Liew and Gardner, 2015).

2.3. DEFORMATION CAPACITY

The CSM employs "base curves" derived on the basis of a regression fit to compression and bending test data for a range of metallic materials, including austenitic and duplex stainless steel. These base curves define the relationship between the deformation capacity (expressed in terms of the strain ratio, $\varepsilon_{csm}/\varepsilon_y$, and the full cross-section slenderness, λ_l, as given by Equations A-2-3 and A-2-4 for plated sections and round HSS, respectively. Each base curve contains two parts: the first part applies to nonslender sections with $\lambda_l \le 0.68$ for plated sections and $\lambda_l \le 0.30$ for round HSS, corresponding to strain ratios, $\varepsilon_{csm}/\varepsilon_y$, greater than or equal to unity, while the second part applies to slender sections with $\lambda_l > 0.68$ for plated sections and $\lambda_l > 0.30$ for round HSS, corresponding to strain ratios, $\varepsilon_{csm}/\varepsilon_y$, less than unity. The two parts meet at the yield slenderness limit, which is the transition point between slender and nonslender sections. Two limits are applied to the strain ratio, $\varepsilon_{csm}/\varepsilon_y$, for nonslender cross sections. The first limit of Λ is to prevent excessive strains, while the second limit, which is related to the adopted elastic, linear hardening material model, is to avoid over-prediction of the material strength. The limit of $\Lambda = 5$ given in Section 2.3.2 only applies when conducting cross-section strength for design by second-order elastic analysis with imperfections according to Appendix 1. If the CSM is utilized in combination with Chapter C and within the limits of Table A-2.1.1 (i.e., stocky members), then a limit of $\Lambda = 15$ is allowed.

The local cross-section slenderness, λ_l, quantifies the susceptibility of a cross section to local buckling. The elastic buckling stress of the cross section, F_{el}, may be calculated through numerical methods, for example, using the CUFSM (Li and Schafer, 2010) or, for I-sections and square and rectangular HSS, using the simplified expressions of Gardner et al. (2019) presented in the Commentary to Appendix 1.

2.4. TENSILE STRENGTH

When calculating the CSM tensile strength of stainless steel structural members, the maximum tensile stress the cross section can reach is determined from the CSM material model given in Figure A-2.2.1 by limiting the strain to the minimum of $15\varepsilon_y$ and $0.10\left(1-F_y/F_u\right)$ (de J. dos Santos et al., 2021). These are the same strain limits used for members in compression that are not affected by cross-sectional and global instabilities.

For tensile members with holes, the provisions for the verification of the net section are the same as those included in Section D2. Due to the large ductility and strong strain hardening exhibited by stainless steels, the attainment of the tensile strength may require a significant amount of deformation. For this reason, a User Note was included in Appendix 2, Section 2.4, indicating that the verification of the net section can be performed at a stress below the tensile strength. The stress in the net section can be calculated based on the CSM material model and the maximum permissible elongation of the member by using the simple expression given in the User Note of Section D2. Alternatively, the more accurate material model given in Appendix 7 may be used.

2.5. COMPRESSIVE STRENGTH

After the maximum attainable strain, ε_{csm}, and the corresponding strain hardening modulus, E_{sh}, are determined, the CSM design stress, f_{csm}, for stainless steel cross-sections under pure compression can be calculated in accordance with Equation C-A-2-1 for nonslender sections with design strains greater than or equal to the yield strain ($\varepsilon_{csm}/\varepsilon_y \geq 1$) and Equation C-A-2-2 for slender sections with design strains less than the yield strain ($\varepsilon_{csm}/\varepsilon_y < 1$). Note that in Equation C-A-2-1 the strain hardening behavior of the material is accounted for through the strain hardening modulus, E_{sh}. The CSM design stress, f_{csm}, can then be employed directly to obtain the nominal compressive strength, P_n, for the limit state of yielding or local buckling using Equation C-A-2-3.

$$f_{csm} = F_y + E_{sh}\,\varepsilon_y\left(\frac{\varepsilon_{csm}}{\varepsilon_y} - 1\right) \text{ for } \varepsilon_{csm}/\varepsilon_y \geq 1.0 \tag{C-A-2-1}$$

$$f_{csm} = E\varepsilon_{csm} \text{ for } \varepsilon_{csm}/\varepsilon_y < 1.0 \tag{C-A-2-2}$$

$$P_n = f_{csm}A_g \tag{C-A-2-3}$$

2.6. FLEXURAL STRENGTH

The CSM design equations for flexural strength were derived by Gardner et al. (2011), Afshan and Gardner (2013), Buchanan et al. (2016), and Zhao et al. (2017), where extensive validation and comparisons with experimental and numerical data are presented. The provisions for asymmetric and singly symmetric cross sections in bending about an axis that is not one of symmetry were developed and evaluated by Zhao and Gardner (2018).

Bending about an Axis of Symmetry. Once the maximum attainable strain in the cross section, ε_{csm}, is determined in accordance with Section 2.3, the design stress distribution can then be derived based on the elastic, linear hardening material model.

For slender sections with a design strain ratio less than unity ($\varepsilon_{csm} / \varepsilon_y < 1$), there is an elastic, linear-varying stress distribution and no benefit arises from strain hardening; the CSM flexural strength is thus directly calculated, as the elastic bending moment resistance multiplied by the strain ratio, as given by Equation A-2-15, for bending about the major or minor axis.

For nonslender sections with a design strain ratio greater than or equal to unity ($\varepsilon_{csm} / \varepsilon_y \geq 1$), the CSM flexural strength was firstly derived analytically through integration of the design stress distribution throughout the cross-section depth, and then transformed into a simplified design formula, as given by Equation A-2-16, where α is the CSM bending coefficient, related to cross-section shape and axis of bending, as shown in Table A-2.6.1.

Bending about an Axis that is Not One of Symmetry. For asymmetric and singly symmetric cross sections in bending about an axis that is not one of symmetry, the maximum attainable compressive strain, $\varepsilon_{csm,c}$, should be determined in accordance with Equations A-2-3 and A-2-4 (i.e., $\varepsilon_{csm,c} = \varepsilon_{csm}$), while the corresponding outer-fiber tensile strain, $\varepsilon_{csm,t}$, should be determined assuming a linearly varying through-depth strain distribution using Equation C-A-2-4, where h is the overall height of the section in the direction of bending and y_c is the distance from the elastic neutral axis (ENA) to the outer compressive fiber:

$$\varepsilon_{csm,t} = \varepsilon_{csm,c}\,\frac{h - y_c}{y_c} \tag{C-A-2-4}$$

When computing $\varepsilon_{csm,t}$ the design neutral axis should first be assumed to be located at the ENA location. If the maximum design strain, $\varepsilon_{csm,max}$, taken as the maximum of $\varepsilon_{csm,c}$ and $\varepsilon_{csm,t}$, is less than the yield strain, ε_y, the use of the ENA may be considered to be appropriate, and the nominal flexural strength should be calculated from Equation A-2-15, with $\varepsilon_{csm} = \varepsilon_{csm,max}$. If the maximum design strain, $\varepsilon_{csm,max}$, is greater than or equal to the yield strain, ε_y, the design neutral axis should be changed from the previously assumed ENA to the location dictated by cross-section equilibrium or, as an approximation, the midpoint between the elastic and plastic neutral axes. $\varepsilon_{csm,t}$ and $\varepsilon_{csm,max}$ should then be recalculated using the new design neutral axis, and the corresponding CSM nominal flexural strength determined using Equation A-2-16.

It is worth pointing out that cross sections bent about a nonsymmetric axis, in which the neutral axis is closer to the compression side, may benefit from strain hardening even if failure due to local buckling takes place before the compression side of the cross section reaches the yield stress, as some yielding may be attained on the tension side at the fibers most distant from the neutral axis.

2.7. COMBINED FLEXURE AND COMPRESSION

The CSM was extended by Zhao et al. (2015, 2016a, 2016b), to account for I-shaped members, as well as square, rectangular, and round HSS subjected to combined compression and bending. Section 2.7 makes use of the combined loading equations included in Chapter H (as well as Equation A-2-17) but anchored to the CSM end-points for the compressive and flexural strengths.

APPENDIX 3
FATIGUE

When the limit state of fatigue is a design consideration, its severity is most significantly affected by the number of load applications, the magnitude of the stress range, and the severity of the stress concentrations associated with particular details. Issues of fatigue are not normally encountered in building design; however, when encountered, and if the severity is great enough, fatigue is of concern and all provisions of this appendix must be satisfied.

Fatigue tests on austenitic and duplex stainless steel have shown that similar crack growth behavior occurs in carbon and stainless steel details and that stainless steels have fatigue strengths very similar to those in carbon steels (Koskimäki and Niemi, 1995; European Commission, 2002, 2008). International design standards therefore specify that the fatigue design rules for carbon steels are applicable to austenitic and duplex stainless steels (CEN, 2005; DNVGL, 2019).

The provisions in this appendix on fatigue strengths apply to structures operating under normal atmospheric conditions and with sufficient corrosion protection and regular maintenance. The effect of seawater corrosion is not covered. Microstructural damage from high temperatures (>300°F or >150°C) is not covered. Furthermore, almost all the fatigue tests on stainless steel joints in the literature had been performed in air. In the presence of a corrosive environment, fatigue strength is reduced, and the magnitude of reduction depends on materials, environment, loading frequency, and other factors. However, the weld geometry has been shown to have a more detrimental effect on fatigue than seawater (European Commission, 2008).

In view of the above, the provisions in this appendix were taken from the 2016 AISC *Specification* with the following minor modifications. Further information regarding the development of these provisions can be found in the Commentary to the 2016 AISC *Specification*.

3.3. PLAIN MATERIAL AND WELDED JOINTS

1. Allowable Stress Range

Some of the fatigue design parameters given in the 2016 AISC *Specification* were not included in Table A-3.1, and some descriptions were modified. This was either because the design of the details was not covered in this Specification, or are not relevant to stainless steel.

2. Specific Requirements for Welded Connections in Cyclically Loaded Structures

As with all welded stainless steel components, AWS D1.6/D1.6M applies. AWS D1.6/D1.6M, clause 4.14, provides specific additional criteria for cyclically loaded welded stainless steel components that address continuity and prohibited weld details. Intermittent fillet welds are prohibited, except on opposite sides of a common plane around a common corner at the discretion of the engineer of record (per AWS D1.6/D1.6M, clause 4.4.4.3).

AWS D1.6/D1.6M, clauses 4.9 and 4.10, provide specific additional criteria for cyclically loaded welded stainless steel components that address transitions. For butt joints in non-HSS cyclically-loaded connections and HSS cyclically loaded axial members, transitions in thickness or width should not exceed 40% with the surface or edge of either part. Alternatively, a non-HSS width transition can occur with a 2-ft (0.6-m) minimum radius tangent to the narrower part of the center of the butt joints.

3.4. BOLTS AND THREADED PARTS

In Appendix 3, Section 3.4(b), "high-strength bolts, common bolts" was replaced by "bolts."

3.5. FABRICATION AND ERECTION REQUIREMENTS FOR FATIGUE

Additional requirements from AWS D1.6/D1.6M were added concerning welds in cyclically loaded members.

3.6. NONDESTRUCTIVE EXAMINATION REQUIREMENTS FOR FATIGUE

The acceptance requirements for austenitic stainless steel in AWS D1.6/1.6M, clauses 8.12.2 or 8.13.2, are the same as those in AWS D1.1/D1.1M, clauses 6.12.2 or 6.13.2. Although no equivalent requirements are given for duplex stainless steel in AWS D1.6/1.6M, the same requirements can be deemed to apply.

APPENDIX 4
STRUCTURAL DESIGN FOR FIRE CONDITIONS

4.1. GENERAL PROVISIONS

This Specification adopts the same requirements included in the 2016 AISC *Specification* for Appendix 4, Sections 4.1.1 to 4.1.4.

5. Avoidance of Embrittlement Due to Contact with Molten Zinc

Zinc can cause a phenomenon called liquid metal embrittlement (LME) of stainless steel, resulting in potentially catastrophic failure (Solo et al., 2017). If galvanized steel is used in the proximity of stainless steel and a fire occurs, the stainless steel should be examined and evaluated for LME caused by the zinc.

4.2. STRUCTURAL DESIGN FOR FIRE CONDITIONS BY ANALYSIS

1. Design-Basis Fire

This Specification adopts the same requirements included in the 2016 AISC *Specification.*

2. Temperatures in Structural Systems under Fire Conditions

The temperature rise in an unprotected stainless steel section can be determined in the same way as for carbon steel:

$$\Delta T_s = \frac{a}{c_s\left(\dfrac{W}{D}\right)}\left(T_F - T_S\right)\Delta t \tag{C-A-4-1}$$

where

D = heat perimeter, in. (m)

T_F = temperature of the fire, °F (°C)

T_S = temperature of the steel, °F (°C)

W = weight (mass) per unit length, lb/ft (kg/m)

a = heat transfer coefficient, Btu/(ft^2-s-°F) (W/m^2-°C)

$= a_c + a_r$ (C-A-4-2)

a_c = convective heat transfer coefficient

a_r = radiative heat transfer coefficient, given as:

$$= \frac{S_B \varepsilon_F}{T_F - T_S}\left(T_{FK}^4 - T_{SK}^4\right) \tag{C-A-4-3}$$

S_B = Stefan-Boltzmann constant

$= 3.97 \times 10^{-14}$ Btu/ft-in-s-°F^4 $\quad (5.67 \times 10^{-8}$ W/m^2-°C$^4)$

$T_{FK} = (T_F + 459)/1.8$ for T_F in °F

$= (T_F + 273)$ for T_F in °C

$T_{SK} = (T_S + 459)/1.8$ for T_S in °F
$= (T_S + 273)$ for T_S in °C
ε_F = emissivity of the fire and view coefficient, which can be taken as 0.4 for stainless steel
c_s = specific heat of the steel, Btu/lb-°F (J/kg-°C)
Δt = time interval, s

For the standard exposure, the convective heat transfer coefficient, a_c, can be approximated as 1.02×10^{-4} Btu / (ft-in.-s-°F) (25 W/m^2-°C).

The specific heat of austenitic and duplex stainless steel, c_s, may be determined from the following with T_S in °F for Equation C-A-4-4 and in °C for Equation C-A-4-4M:

$$c_S = 0.107 + (0.372 \times 10^{-5})(T_S - 32) - (2.15 \times 10^{-8})(T_S - 32)^2 - (5.49 \times 10^{-12})(T_S - 32)^3, \text{ BTU/(lb-°F)} \tag{C-A-4-4}$$

$$c_S = 450 + 0.280T_S - (2.91 \times 10^{-4})T_S^2 + (1.34 \times 10^{-7})T_S^3, \text{ J/kg-K} \tag{C-A-4-4M}$$

Passive fire protection is very rarely applied to stainless steels.

The Commentary to the 2016 AISC *Specification* gives further background information.

3. Material Strengths at Elevated Temperatures

3a. Thermal Elongation

The data for austenitic stainless steel were taken from a German standard (DIN, 1992), while for duplex steel the data were provided by a stainless steel mill and represent an average value for a range of commonly used duplex alloys.

3b. Mechanical Properties at Elevated Temperatures

For the modeling and design of stainless steel structures in fire, it is necessary to consider the material properties at elevated temperatures. The key parameters are typically expressed as a proportion of the corresponding room temperature properties. Following work by Gardner et al. (2010), generic sets of reduction factors for groups of stainless steel alloys were developed. These sets group together alloys exhibiting similar elevated temperature properties, with the tabulated retention factors derived based on the mean retention factors for each group. These are the same factors as given in the first edition of AISC Design Guide 27 (Baddoo, 2013); however, values for the reduction in strength at the 2% total strain are also included since this parameter is used in the material model at elevated temperatures given in Appendix 7.

Stainless steel exhibits a pronounced response to cold work, and tests were carried out on cold-formed stainless steel sections by Ala-Outinen (1996) and Chen and Young (2006) to evaluate the response at elevated temperatures. It was shown that the increased material strength existing at room temperature due to cold work is constant up to about 1,300°F (700°C), after which the beneficial effects begin to decrease, and the influence of cold forming totally disappears at 1,650°F (900°C). Considering the cold-work condition, where cold forming associated strength enhancements are utilized in design, for temperatures of 1,400°F (760°C) and above, the annealed 0.2% proof strength retention factors must be reduced by 20%. Similarly, for temperatures of 1,400°F (760°C) and above, the annealed 2% proof strength retention factors must be reduced by 10%. The tensile strength at elevated temperatures is less sensitive to the effects of cold work and therefore the annealed retention factors may be used for all elevated temperatures (Gardner et al., 2010).

Table C-A-4.2.1 gives some mechanical properties at elevated temperature for precipitation hardening alloy S17400 in the H1150 heat treatment condition (ASME, 2019). Strength data at higher temperatures is given in the *Aerospace Structural Metals Handbook* (Brown and Setlak, 2004).

4. Structural Design Requirements

This Specification adopts the same requirements included in the 2016 AISC *Specification* for Appendix 4, Sections 4.2.4a, 4.2.4b, and 4.2.4c.

4d. Design by Simple Methods of Analysis

Simple methods may suffice when a structural member or component can be assumed to be subjected to uniform heat flux on all sides and the assumption of a uniform temperature is reasonable as, for example, in a free-standing column surrounded by fire.

This Specification adopts the same approach used in the 2016 AISC *Specification* for evaluating the performance of individual members at elevated temperatures during exposure to fire, where the nominal strength of the member at elevated temperatures, R_n, is determined from Chapters D through H, and Appendix 4, using material strength and stiffnesses at elevated temperatures defined in Tables A-4.2.1 and A-4.2.2 for austenitic stainless steel alloys, and Tables A-4.2.3 and A-4.2.4 for duplex stainless steel alloys. The design strength is then calculated as ϕR_n, where ϕ is the resistance factor at ambient temperature given in Chapters D through H. The design of connections at elevated temperatures is not covered in this Specification due to insufficient research data on the degradation of the material properties of stainless steel bolts or welds at elevated temperatures.

TABLE C-A-4.2.1
Properties of Precipitation Hardening Type S17400 at Elevated Temperatures

Steel Temperature, °F (°C)	$k_y = F_y(T)/F_y$	$k_u = F_u(T)/F_u$
68 (20)	1.00	1.00
200 (93)	0.92	1.00
400 (200)	0.85	0.97
500 (260)	0.83	0.95
600 (320)	0.81	0.94
750 (400)	0.77	0.90
800 (430)	0.76	0.87
1000 (540)	0.57	0.66

For compression and flexural members susceptible to buckling instabilities, the provisions given in Chapters E and F of this Specification (at ambient temperature), with steel properties (E, F_y, and F_u) reduced for elevated temperatures, can considerably overestimate the strength of the member. To address this, the flexural buckling provisions for nonslender columns given in Chapter E, and the lateral-torsional buckling provisions for compact I-shaped members and channels given in Chapter F, were slightly modified based on the results from detailed finite element method analyses on I-shaped and rectangular HSS columns, and I-shaped beams susceptible to global instabilities at elevated temperature reported by Kucukler et al. (2020). The design under fire conditions of compression members with slender elements and flexural members with noncompact and slender elements is not covered in this Specification.

For compression members under fire conditions, the flexural buckling provisions in Chapter E were modified by replacing the flexural coefficients α and β_2 given in Table E3.1 by the temperature dependent coefficients $\alpha(T)$ and $\beta_2(T)$ given in Table A-4.2.5. Excluding the difference in the definition of the flexural buckling coefficients and material factors used to account for the material properties degradation at elevated temperatures, the expressions given in this section for calculating the flexural buckling strength of columns under fire conditions are essentially the same as those given in Chapter E for ambient temperature. The format of the expression to calculate the critical stress for the limit state of inelastic flexural buckling in Chapter E (Equation E3-3) was modified in this section so that it is given as a function of $\beta_2(T)$ (Equation A-4-2). The temperature dependent coefficients $\alpha(T)$ and $\beta_2(T)$ converge to the coefficients α and β_2 when the temperature approaches room temperature, allowing for continuity between the flexural buckling curve at elevated temperature and the one given in Chapter E for room temperature.

For flexural members under fire conditions, the only modifications to the lateral-torsional buckling provisions given in Chapter F are the definitions of the lateral-torsional buckling coefficients, β_{LT} and $\beta_{p,LT}$, given in Table F2.1, which in this section are replaced with the temperature dependent coefficients $\beta_{LT}(T)$ and $\beta_{p,LT}(T)$, as given in Table A-4.2.6. The expressions given in Table A-4.2.6 for the temperature dependent coefficient $\beta_{LT}(T)$ were derived based on the results from numerical models on austenitic and duplex stainless steel I-shaped beams at elevated temperatures, while the expressions for the temperature dependent coefficient $\beta_{p,LT}(T)$ were derived by considering that β_{LT} is implicitly included in $\beta_{p,LT}$. Similar to the case of compression members, the temperature dependent coefficients $\beta_{LT}(T)$ and $\beta_{p,LT}(T)$ converge to the coefficients β_{LT} and $\beta_{p,LT}$ when the temperature approaches the room temperature, allowing for continuity between the lateral-torsional buckling curves at elevated temperature and those given in Chapter F for room temperature.

4.3. DESIGN BY QUALIFICATION TESTING

This Specification adopts the same requirements included in the 2016 AISC *Specification*.

APPENDIX 5
EVALUATION OF EXISTING STRUCTURES

5.1. GENERAL PROVISIONS

The load combinations referred to in this chapter pertain to static loading because it is the most prevalent condition encountered. If other loading conditions are a consideration, the appropriate load combination from ASCE/SEI 7 (ASCE, 2016) or from the applicable building code should be used.

5.2. MATERIAL PROPERTIES

1. Determination of Required Tests

The extent of tests required depends on the nature of the project, the criticality of the structural system or member evaluated, and the availability of records pertinent to the project. Thus, the engineer of record has the responsibility to determine the specific tests required and the locations from which specimens are to be obtained.

2. Tensile Properties

Samples required for tensile tests should be removed from regions of reduced stress, such as at flange tips at beam ends and external plate edges, to minimize the effects of the reduced area. The number of tests required will depend on whether they are conducted to merely confirm the strength of a known material or to establish the strength of some other material.

4. Base Metal Notch Toughness

The engineer of record should specify the location of samples. Samples should be cored, flame cut, or saw cut. The distance from the edge of flat tension specimens [generally, specimens ½ in. (13 mm) thick or less] need to be made only large enough to obtain the grip width. The distance from the center of a cylindrical tension specimen to either of the thermal cut edges should be 1 in. (25 mm) or larger. The engineer of record will determine if remedial actions are required, such as the possible use of bolted splice plates.

5. Weld Metal

Because connections typically have a greater reliability index than structural members (see Commentary Section B3.1), strength testing of weld metal is not usually necessary. The specified provisions in AWS D1.6/D1.6M provide a means for judging the quality of welds. Where feasible, any samples removed should be obtained from compression splices rather than tension splices, because the effects of repairs to restore the sampled area are less critical.

6. Bolts

Because connections typically have a greater reliability index than structural members (see Commentary Section B3.1), removal and strength testing of fasteners is not usually necessary. However, strength testing of bolts is required where they cannot be properly identified otherwise.

5.3. EVALUATION BY STRUCTURAL ANALYSIS

2. Strength Evaluation

Resistance and safety factors reflect variations in determining strength of members and connections, such as uncertainty in theory and variations in material properties and dimensions. If an investigation of an existing structure indicates that there are variations in material properties or dimensions significantly greater than those anticipated in new construction, the engineer of record should consider the use of more conservative values.

5.4. EVALUATION BY LOAD TESTS

1. Determination of Load Rating by Testing

This Specification adopts the same requirements included in the 2016 AISC *Specification*.

Stainless steel displays higher ductility and greater strain hardening than carbon steel, and therefore, the test rig capabilities may need to be greater than those required for testing carbon steel members of equivalent material yield strength. This not only applies to rig loading capacity, but also to the ability of the rig to allow greater deformation of the specimen.

It should be noted that at higher specimen loads, the effects of creep become more manifest, and this may mean that strain or displacement readings do not stabilize within a reasonable time.

2. Serviceability Evaluation

In certain cases, serviceability performance must be determined by load testing. It should be recognized that complete recovery (in other words, return to the pretested deflected shape) after removal of maximum load is unlikely because of phenomena such as local yielding, slip in bolted connections, effects of continuity, etc. Because most structures exhibit some slack when load is first applied, it is appropriate to project the load-deformation curve back to zero load to determine the slack and exclude it from the recorded deformations. Where desirable, the applied load sequence can be repeated to demonstrate that the structure is essentially elastic under service loads and that the permanent set is not detrimental.

5.5. EVALUATION REPORT

Extensive evaluation and load testing of existing structures is often performed when appropriate documentation no longer exists or when there is considerable disagreement about the condition of a structure. The resulting evaluation is only effective if well documented, particularly when load testing is involved. Furthermore, as time passes, various interpretations of the results can arise unless all parameters of the structural performance, including material properties, strength, and stiffness, are well documented.

APPENDIX 6

MEMBER STABILITY BRACING

The expressions for the calculation of member stability bracing in this Specification were taken from the 2016 AISC *Specification*. Information regarding the development of these expressions can be found in the Commentary to the 2016 AISC *Specification.*

A numerical study was performed to assess the strength and stiffness requirements of bracing to stainless steel members. A series of concentrically braced stainless steel columns of varying slenderness were considered. It was shown that provided the bracing stiffness requirements given in the 2016 AISC *Specification* were met, the capacity of a stainless steel braced member reached a similarly high proportion of its fully braced value as an equivalent carbon steel member. However, owing to the nonlinear stress-strain response of stainless steel, greater capacity demands were placed on the braces; hence the strength requirements of the braces given by Equations A-6-1, A-6-3, A-6-5, A-6-7, and A-6-9 were increased by 50% over the values given in the 2016 AISC *Specification*.

APPENDIX 7
MODELING OF MATERIAL BEHAVIOR

This appendix provides analytical expressions for modeling of material behavior, as well as calculating the secant and tangent modulus, of austenitic and duplex stainless steel structural members.

An accurate description of the stress-strain behavior of stainless steel is essential for use in structural design codes, and advanced analytical and numerical models, whose applications may include the simulation of section forming, the structural behavior of members and connections, the response of structures under extreme loads, and so on.

7.1 MATERIAL BEHAVIOR AT AMBIENT TEMPERATURE

1. Stress-Strain Behavior

The stainless steel material model presented in this appendix is based on the original single stage expression developed by Ramberg and Osgood (1943) and modified by Hill (1944), which is given by Equation A-7-1a. This equation requires three independent parameters to be specified in order to describe the stress-strain behavior for a particular stainless steel material. Namely, the initial modulus of elasticity, E, the specified minimum yield strength, F_y, and a strain hardening coefficient, n, which characterizes the degree of nonlinearity of the stress-strain curve (lower values imply a greater degree of nonlinearity), as shown in Figure C-A-7.1.

Recent research by Arrayago et al. (2015) showed that the value of n may be accurately obtained using Equation C-A-7-1, based on the ratio of the 0.05% offset strength, $F_{0.05\%}$, to the specified minimum yield strength, F_y:

$$n = \frac{\ln(4)}{\ln\left(F_y / F_{0.05\%}\right)} \tag{C-A-7-1}$$

Examination of a large collection of measured stress-strain curves on a variety of stainless steel grades and product forms suggested that a representative value of n equal to 7 and 8 can be used for austenitic and duplex stainless steel, respectively.

While the single Ramberg-Osgood formulation gives excellent agreement with experimental stress-strain data up to the specified minimum yield strength, at higher strains the model generally overestimates the stress corresponding to a given level of strain. This led to the development of a number of two-stage (and three-stage) models, notably by Mirambell and Real (2000), Rasmussen (2003), Gardner and Nethercot (2004a), Gardner and Ashraf (2006), and Quach et al. (2008). The use of two adjoining Ramberg-Osgood curves leads to improved modeling accuracy at strains above the specified minimum yield strength. The basic Ramberg-Osgood expression is used up to the 0.2% offset yield strength, then a modified expression redefines the origin for the second curve as the specified minimum yield strength and provides continuity of gradients. The improved accuracy at higher strains of this compound Ramberg-Osgood expression are demonstrated in Figure C-A-7.2.

The two-stage Ramberg-Osgood material model, including recent refinements based on the work of Arrayago et al. (2015), was adopted in this appendix. According to these models, when the specified minimum tensile strength, F_u, of austenitic and duplex stainless steels is not known, it can be obtained according to Equation C-A-7-2, based on the specified minimum yield strength and the initial modulus of elasticity:

$$F_u = \frac{F_y}{0.2 + 185\dfrac{F_y}{E}} \qquad \text{(C-A-7-2)}$$

2. Tangent Modulus, E_t

The expression for the tangent modulus was derived from Equation A-7-1a by computing $d\sigma/d\varepsilon$. Therefore, the expression for the tangent modulus is only valid for stresses below or equal to the specified minimum yield strength.

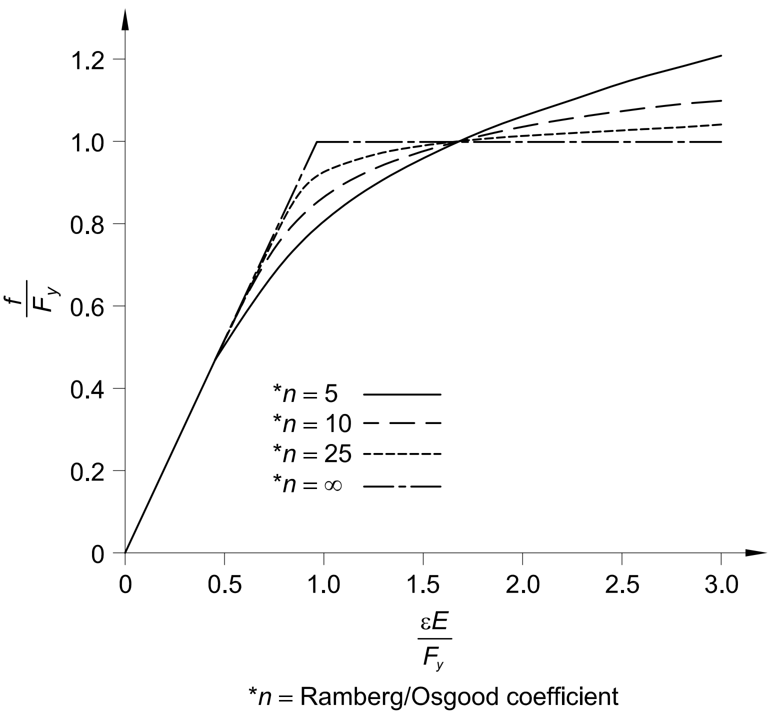


Fig. C-A-7.1. Effect of the n *coefficient on the nonlinearity of the stress-strain curve.*

3. Secant Modulus, E_s

The expression for the secant modulus was also obtained from the modified Ramberg-Osgood equation (Equation A-7-1a) by computing the ratio, f/ε. The expression for the secant modulus is only valid for stresses that are lower or equal to the specified minimum yield stength.

7.2 MATERIAL BEHAVIOR AT ELEVATED TEMPERATURES

Extensive research studies have been carried out to investigate the elevated temperature material response of different stainless steel alloys (Gardner et al., 2010; Liang et al., 2019). Based upon these research studies, it was found that the two-stage Ramberg-Osgood stress-strain relationship very accurately represents the material response of different stainless steel alloys at elevated temperatures, also enabling consistency with the two-stage Ramberg-Osgood material stress-strain model at room temperature adopted in this Specification. Another very important advantage of the use of the adopted Ramberg-Osgood material model at elevated temperatures is that it enables direct determination of residual plastic strains resulting from the initial residual stresses that have to be defined in the numerical modeling of welded stainless steel elements in fire. The development of the adopted material model at elevated temperatures is described in Gardner et al. (2016) and Liang et al. (2019).

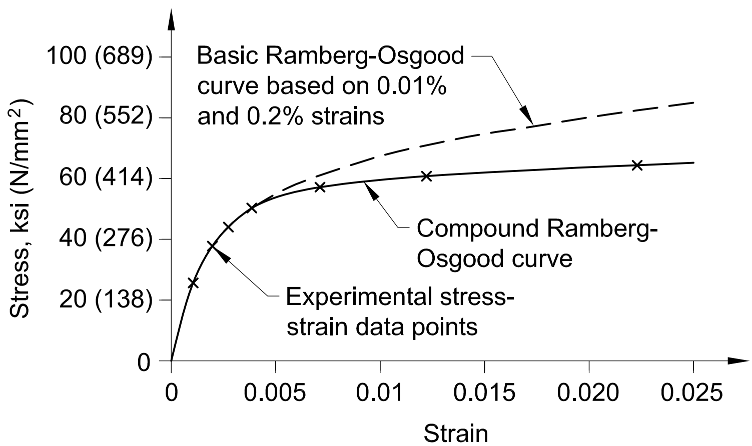


Fig. C-A-7.2. Comparison between compound and basic Ramberg-Osgood models.

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Metric Conversion Factors for Common Steel Design Units Used in AISC Specifications

Unit	Multiply	by	to obtain
length	inch (in.)	25.4	millimeters (mm)
length	foot (ft)	0.3048	meters (m)
mass	pound-mass (lbm)	0.4536	kilograms (kg)
stress	ksi	6.895	megapascals (MPa), N/mm^2
moment	kip-in.	113 000	N-mm
energy	ft-lbf	1.356	joule (J)
force	kip (1 000 lbf)	4 448	newton (N)
force	psf	47.88	pascal (Pa), N/m^2
force	plf	14.59	N/m
temperature	To convert °F to °C: $t_c° = (t_f° - 32)/1.8$		

force in lbf or N = mass × g
where g, acceleration due to gravity = 32.2 ft/sec^2 = 9.81 m/sec^2

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