NOTE: Revisions made since the previous public review period (May 1-June 15, 2020) are indicated with underline/strikethrough.

Specification for Structural Stainless Steel Buildings

Public Review Draft dated October 14, 2020

AMERICAN INSTITUTE OF STEEL CONSTRUCTION
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Chicago, Illinois 60601-6204
PREFACE

(This Preface is not part of ANSI/AISC 370-XX, 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, 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.

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.

The Committee gratefully acknowledges Matt Smith, Board Oversight, and the following advisory members, Javier Avila Mendoza, Nancy Baddoo, Leroy Gardner, and Ben Young, 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 right-hand column of the list identifies the first time the symbol is used in this document. Symbols without text definitions are omitted.

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| \( M_r \) | 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 |
| \( M_{r,b} \) | 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) | \( \) | \( H3 \) |
| \( M_y \) | Yield moment about the axis of bending, kip-in. (N-mm) | \( \) | \( F4.1 \) |
| \( M_{y,k} \) | Effective moment at the end of the unbraced length opposite from \( M_y \), kip-in. (N-mm) | \( \) | App. 1.3.2b |
| \( M_t \) | Smaller moment at end of unbraced length, kip-in. (N-mm) | \( \) | App. 1.3.2b |
| \( M_t' \) | 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 |
| \( P_{o} \) | 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 |
| \( P_{t} \) | 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 |
| \( P_{r} \) | 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 2.2 D2(b), kips (N) | \( \) | H3 |
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| \( P_{o} \) | Cross-section compressive strength, kips (N) | \( \) | C2.3 |
| \( P_{b} \) | Nominal bearing strength, kips (N) | \( \) | J9 |
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| \( P_{r} \) | 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) | \( \) | \( \) |
| \( P_{r} \) | 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 |
| \( R_{o} \) | Required strength using ASD load combinations | \( \) | B3.2 |
| \( R_{re} \) | Reduction factor for joints using a pair of transverse fillet welds only | \( \) | \( \) |
| \( R_{n} \) | Nominal strength | \( \) | B3.3 |

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$R_e$ Combined strength of fillet weld group, kips (N) ......................... J2.4
$R_n$ Nominal strength of the connected material, kips (N) .................. J3.10
$R_{nul}$ Total nominal strength of longitudinally loaded fillet welds, as determined in accordance with Table J2.54, kips (N) ......................... J2.4
$R_{nt}$ Total nominal strength of transversally loaded fillet welds, as determined in accordance with Table J2.54 without the increase in Section J2.4(b), kips (N) ......................... J2.4
$R_p$ Web plastification factor ......................................................... F4.1
$R_{sy}$ Bending strength reduction factor ......................................... F5.2
$R_{fl}$ Reduction factor for reinforced or nonreinforced transverse partial-joint penetration (PJP) groove welds ........................................ App. 3.3
$R_i$ 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
$S_n$ Nominal snow load, kips (N) .................................................... App. 4.1.4
$S_e$ Effective section modulus taken 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
$S_y$ 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,$^\circ$F ................................................................. App. 4.2.4d
$T_n$ Required tension force using ASD load combinations, kips (kN) .... J3.9
$T_e$ Available torsional strength determined in accordance with Section H2.1, kip-in. (N-mm) .......................................................... H2.2
$T_r$ Required torsional strength, determined in accordance with Chapter C, using LRFD or ASD load combinations, kip-in. (N-mm) .... H2.2
$T_b$ Minimum fastener tension for stainless steel bolts, kips (N) ............ J3.8
$T_n$ Required tension force using LRFD load combinations, kips (kN) .... J3.9
$U$ Shear lag factor .................................................................... D3
$V_{br}$ Required shear strength of the bracing system, kips (N) ............. App. 6.2.1
$V_{br}$ Available shear strength, kips (N) .............................................. H2.2
$V_{cl}$ Available shear strength, kips (N) .............................................. G2.3
$V_{cl}$ Available shear strength, kips (N) .............................................. G2.3
$V_n$ Nominal shear strength, kips (N) .................................................. G1
$V_f$ Required shear strength in the panel being considered, kips (N) ....... G2.3
$V_{fi}$ Required shear strength, determined in accordance with Chapter C, using LRFD or ASD load combinations, kips (N) ...................... H2.2
$V_i$ Gravity load applied at level $i$ from the LRFD load combination or ASD load combination, as applicable, kips (N) ............... C2.2b
$V_i$ 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
$\alpha$ Clear distance between transverse stiffeners, in. (mm) .................... G2.1
$\alpha$ Distance between connectors, in. (mm) ........................................ E6.1
$\alpha$ Shortest distance from edge of pin hole to edge of member measured parallel to the direction of force, in. (mm) ......... D5.1
$\alpha_f$ Ratio of the flange area to the gross area of member .................. App. 2.7.1
$\alpha_w$ Ratio of the web area to the gross area of member .................... App. 2.7.1

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Factor for fillers .............................................................. J3.8
Distance between flange centroids, in. (mm) ......................... F2.2
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
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)...........
Plate buckling coefficient .................................................. E7.1
Web plate shear buckling coefficient ........................................ G2.1
Actual length of end-loaded weld, in. (mm) ............................. J2.2b
Length of connection, in. (mm) ............................................ Table D3.1
Length of bearing, in. (mm) ................................................. J11
Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material, in. (mm) ....

Distance from the near side of the connecting branch to end of chord, in. (mm) ................................................................. K1.1
Overlap length measured along the connecting face of the chord beneath the two branches, in. (mm) ............................................. K2.1
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Strain hardening coefficient ................................................ App. 7.1.1
Number of braced points within the span............................... App. 6.3.2a
Threads per inch (per mm) .................................................. App. 3.4
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Auxiliary coefficient ......................................................... C2.3
Number of slip planes required to permit the connection to slip ...... J3.8
Number of stress range fluctuations in design life .................... App. 3.3
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Pitch, in. per thread (mm per thread) ........................ .......... App. 3.4
Minimum value of the center-to-center spacing of bolts in the direction of stress, in. (mm) ..................................................... J3.10
Minimum value of the center-to-center spacing of bolts normal to the direction of stress, in. (mm) ..................................................... J3.10
Radius of gyration, in. (mm) ............................................... E2
Internal radius of corner, in. (mm) ....................................... J12
Retention factor depending on bottom flange temperature ....... App. 4.2.4d
Radius of gyration about the geometric axis parallel to the connected legs, in. (mm) ................................................................. E5
Internal corner radii, in. (mm) .............................................. B4.32
Minimum radius of gyration of individual component, in. (mm) .... E6.1
Polar radius of gyration about the shear center, in. (mm) .......... E4
Effective radius of gyration for lateral-torsional buckling, in. (mm) F4.2
Radius of gyration about the x-axis, in. (mm) ................. E4
Radius of gyration about y-axis, in. (mm) ............................... E4
Longitudinal center-to-center spacing (pitch) of any two consecutive holes, in. (mm) ................................................................. E4.2b
Design wall thickness of round HSS, in. (mm) .......................... B4.1b
Design thickness of plate, in. (mm) ...................................... D5.1
Thickness of wall, in. (mm) ............................................... E7.2
Design thickness of plate, in. (mm) ...................................... G4
Design thickness corresponding to longer side, in. (mm) .......... G7.1
Thickness of connected material, in. (mm) .............................. J3.10
Total thickness of fillers, in. (mm) ......................................... J5.3
Design wall thickness of HSS main member, in. (mm) ............. K2.1
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1078 $\varepsilon(T)$ Engineering strain at elevated temperatures ....................... App. 7.2
1079 $\varepsilon_u(T)$ Ultimate strain at elevated temperatures ......................... App. 7.2
1080 $\varepsilon_y(T)$ Strain at the yield stress at elevated temperatures .......... App. 7.2
1081 $\gamma$ 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
1082 $\zeta$ 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
1083 $\lambda$ Width-to-thickness ratio for the element ....................................... E7.1
1084 $\lambda_1$ Local cross-section slenderness .................................................. App. 1.3.3d
1085 $\lambda_1'$ Cross-section slenderness ............................................................. App. 2.1
1086 $\lambda_{ef}$ Limiting width-to-thickness ratio for compact flange ............... F7.2
1087 $\lambda_e$ Limiting width-to-thickness ratio as defined in Table B4.1a ........ E7.1
1088 $\lambda_{ef}$ Limiting width-to-thickness ratio for noncompact flange ........... F7.2
1089 $\mu$ Mean slip coefficient ......................................................................... J3.8
1090 $\nu$ Poisson’s ratio = 0.3 ........................................................................ E7.1
1091 $\phi$ Resistance factor ............................................................................. B3.1
1092 $\Omega$ Safety factor .................................................................................. B3.2
1093 $\Lambda$ Upper bound strain limit .................................................................. App. 1.3.3d
1094 $\rho_{csm}$ Reduction factor ........................................................................ App. 1.3.3e
1095 $\rho_{w}$ Maximum shear ratio within the web panels on each side of the transverse stiffener ................................................................. G2.3
1096 $\theta$ Angle between the line of action of the required force and the weld longitudinal axis, degrees ......................................................... J2.4
1097 $\theta$ Acute angle between the branch and chord, degrees ....................... K2.1
1098 $\tau_b$ General angle between the branch and chord, degrees ................. K2.1
1099 $\tau_g$ General stiffness reduction factor .................................................... App. 1.2.2b
1100 $\tau_{ef}$ General stiffness reduction factor .................................................... C2.3
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, 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 a specification that requires one or more of the following elements, by mass percent, to have a minimum content equal to or greater than 0.30 for aluminum, 0.0008 for boron, 0.30 for chromium, 0.30 for cobalt, 0.40 for copper, 0.40 for lead, 1.65 for manganese, 0.08 for molybdenum, 0.30 for nickel, 0.06 for niobium (columbium), 0.60 for silicon, 0.05 for titanium, 0.30 for tungsten (wolfram), 0.10 for vanadium, 0.05 for zirconium, or 0.10 for any other alloying element, except sulphur, phosphorus, carbon, and nitrogen.

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.

Autopassivating. Protective passive metal oxide film that stainless steel forms spontaneously on exposure to air or moisture as long as the surface is free of exogenous surface contamination.

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 structural 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.

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

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

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

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

Buckling strength. Strength for instability limit states.

Built-up member. Member fabricated from structural stainless steel components, which may include rolled or extruded sections, built-up sections, and/or plates, using intermittent welds or fasteners. Cross section, section, shape. Member, cross section, section or shape fabricated from structural stainless steel elements that are welded (by conventional or laser/laser hybrid welding methods) or bolted together.

Built-up section (or shape). Section fabricated from structural 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 that conforms to a specification that requires, by heat analysis in mass, a maximum of 2% carbon, 1.65% manganese, 0.60% silicon and 0.60% copper in addition to much smaller amounts of other elements.


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.

Cladding. Exterior covering of structure.

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.

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

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

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

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

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

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

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

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, φRn.

Design thickness. Reduction in nominal thickness used in design to account for an expected thickness less than the nominal thickness based on a relevant ASTM standard.

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

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

Diaphragm plate. Plate possessing in-plane shear stiffness and strength, used to transfer forces to the supporting elements.

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

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

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

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

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

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

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

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 contamination. Oxidized ferrous 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, and even 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. See also AWS D1.6/D1.6M.

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.

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

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

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

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

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

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 1-1/2 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 equilibrated 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.

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

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

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.

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

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

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

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

Nominal 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.

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

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

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

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

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

Passivation. Treatment for corrosion-resistant steel to eliminate corroducible 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 capture (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.

Postweld heat treatment. Any heat treatment after welding

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 WPSs 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, \( \phi \). 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, \( \Omega \). 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 \( P-\delta \) effect and \( P-\Delta \) effect.

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

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

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

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

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

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

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

Shear wall†. Wall that provides resistance to lateral loads in the plane of the wall and provides stability for the structural system.
Shear yielding (punching). In an HSS connection, limit state based on out-of-plane shear strength of the chord wall to which branch members are attached.

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

Sideways buckling (frame). Stability limit state involving lateral sideways instability of a frame.

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

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

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

Slender-element section. 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 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.

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

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

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

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

Transverse 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.

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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.

- **AHJ** (authority having jurisdiction)
- **AISC** (American Institute of Steel Construction)
- **AISI** (American Iron and Steel Institute)
- **ANSI** (American National Standards Institute)
- **ASCE** (American Society of Civil Engineers)
- **ASD** (allowable strength design)
- **ASME** (American Society of Mechanical Engineers)
- **ASNT** (American Society for Nondestructive Testing)
- **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)
- **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)
- **LRFD** (load and resistance factor design)
- **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)
- **SEI** (Structural Engineering Institute)
- **SFPE** (Society of Fire Protection Engineers)
- **SMAW** (shielded metal arc welding)
- **SWI** (senior welding inspector)
- **SWPS** (Standard welding procedure specification)
- **UNC** (Unified National Coarse)
- **UNS** (unified numbering system)
- **UT** (ultrasonic testing)
Welding Inspector (WI)
Welder Performance Qualification Records (WPQR)
Welding Procedure Specification (WPS)
CHAPTER A

GENERAL PROVISIONS

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

The chapter is organized as follows:

A1. Scope
A2. Referenced Specifications, Codes, and Standards
A3. Material
A4. Structural Design Documents and Specifications

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

User Note: Guidance on the choice of stainless steel for a range of service conditions is given in AISC Design Guide 27.

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 the structural stainless steel systems, where the stainless steel elements are defined in Section 2.1 of the AISC Code of Standard Practice for Structural Stainless Steel Buildings (AISC 313).

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.

User Note: This Specification is required because of the differences in the mechanical and physical properties of stainless steels are different from between the stainless steels covered by this Specification and the those of the carbon other steel alloys covered by the AISC Specification for Structural Steel Buildings (ANSI/AISC 360-16). Unlike carbon steel, stainless steel alloys are identified by the Unified Numbering System (UNS) designation and are typically specified based on the required corrosion resistance. The handling, fabrication, cleaning requirements, and physical and mechanical property differences between each stainless steel alloy family and carbon other steel alloys must be considered during design. The requirements for welding or fastening stainless steel to carbon other steel alloys or other metals in order to avoid galvanic corrosion are also given in this Specification.

This Specification sets forth 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. The design of carbon steel plate clad with stainless steel is not covered by this Specification.
User Note: User notes are intended to provide concise and practical guidance in the application of the Specification provisions.

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: For the design of cold-formed stainless steel structural members, the provisions in the ASCE Specification for the Design of Cold-Formed Stainless Steel Structural Members (ASCE/SEI 8-21) are available. However, this Specification provides guidance for a) cold-formed hollow structural sections (HSS), b) structural sections built up from cold rolled plate, bar and strip, and c) cold drawn bar products.

This Specification includes the Symbols, the Glossary, Abbreviations, Chapters A through N, and Appendices I 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, must 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).

A2. REFERENCED SPECIFICATIONS, CODES, AND STANDARDS

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

(a) American Institute of Steel Construction (AISC)

AISC 313-21 Code of Standard Practice for Structural Stainless Steel Buildings
ANSI/AISC 360-16 Specification for Structural Steel Buildings
ANSI/AISC N690-18 Specification for Safety-Related Steel Structures for Nuclear Facilities
ANSI/AISC N690s1-15 Specification for Safety-Related Steel Structures for Nuclear Facilities, Supplement No. 1

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(b) American Iron and Steel Institute (AISI)
  AISI S902-17 Test Standard for Determining the Effective Area of Cold- formed Steel Compression Members

(c) American Society of Civil Engineers (ASCE)
  ASCE/SEI 7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures
  ASCE/SEI 8-21 Specification for the Design of Cold-Formed Stainless Steel Structural Members

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

(e) 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

(f) ASTM International (ASTM)
  A6/A6M-19 Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling
  A182/A182M-19a Standard Specification for Forged or Rolled Alloy and Stainless Steel Pipe Flanges, Forged Fittings, and Valves and Parts for High-Temperature Service
  A193/A193M-17 Standard Specification for Alloy-Steel and Stainless Steel Bolting for High Temperature or High Pressure Service and Other Special Purpose Applications
  A194/A194M-18 Standard Specification for Carbon Steel, Alloy Steel, and Stainless Steel Nuts for Bolts for High Pressure or High Temperature Service, or Both
  A240/A240M-18 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-18a 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
  A370-19e1 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
  A473-19 Standard Specification for Stainless Steel Forgings
  A479/479M-18 Standard Specification for Stainless Steel Bars and Shapes for Use in Boilers and Other Pressure Vessels
  A480/A480M-19 Standard Specification for General Requirements for Flat-Rolled Stainless and Heat-Resisting Steel Plate, Sheet, and Strip
  A484/A484M-19 Standard Specification for General Requirements for Stainless Steel Bars, Billets, and Forgings

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A3. MATERIAL

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 current version of the relevant ASTM standard specification, or standard practice, or standard method(s) listed in Section A3.1b and other requirements of the order.

User Note: At the time of issuance, hollow structural sections, 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 often three to eight weeks.

The most commonly stocked hot-rolled profiles are up to 6 in. (150 mm) in
size, but larger sizes are less commonly inventoried such as C8×18.75
(C203×476) channel and other shapes are readily available. Extruded shapes
are readily available and typically fall within an 8 in. (200-203 mm) diameter,
but that will vary with the alloy and supplier. Laser and laser-hybrid sections
are commonly stocked including sizes of up to 86 in. (203152 mm) angles,
sizes are commonly stocked; beams up to 18-50 in. (1270460 mm) tall, and
channels up to C15×33.945 in. (C380×861 mm); mill orders of and other
standard and custom shapes typically require a three to four week lead time.
Structural shapes welded by other welding processes are also available,
including large and heavy sections.

**User Note:** ASTM stainless steel standards specifications require identification
of alloys by their UNS designation, in accordance with SAE J1086, and a
specific ASTM standard specifies. UNS designations identify a unique
chemistry range but are not a specification and the range is often adjusted
within ASTM standards specifications to meet the requirements of a specific
product. 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**

The EOR shall assess the service environment characteristics that are expected
to affect corrosion performance before specifying the stainless steel alloy. A full
assessment of the service environment shall be done prior to design to determine
appropriate candidate alloys.

**User Note:** Refer to the Commentary for guidance on evaluation of the characteristics of different service environments, AISC Design Guide 27 for information on the selection of appropriate alloys for different service environments, and Section A3.1c for minimum alloy specification requirements for specific service environments. If design work occurs prior to service environment assessment, the wrong alloy(s) may be selected. Refer to the Commentary to Section A3 for guidance on evaluation of the characteristics of different service environments, AISC Design Guide 27 for information on the selection of appropriate alloys for different service environments, and Section A3.1c for minimum alloy specification requirements for specific service environments. Without an assessment of the environment, the alloy selection guidance in AISC Design Guide 27 cannot effectively be used. Inappropriate alloy selection could cause life safety, structural or aesthetic failures.

1b. **ASTM Designations**

The following structural stainless steel alloys and structural components manufactured in accordance with the ASTM specifications standards in this section (A3.1b) are approved for use under this Specification.

Alloy Family: 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 (of chromium carbide precipitation to the grain boundaries) in the heat effected zones adjoining each weld. The other austenitic and duplex stainless steels included in Specifications are low carbon and have no special specification requirement. Also see the Commentary, Table User Note A3.2, and AISC Design Guide 27.

Provisions for precipitation hardening stainless steels are limited to tension members, fittings, and fasteners.

The ASTM standards for stainless steel have more minimum order requirements than the equivalent carbon steel material and product standards. Sections (a) to (h) list the minimum order requirements for each stainless steel product. Finish, dimensional and special tolerances, condition, and other mandatory ordering requirements of the ASTM specifications standards shall be specified as indicated for each product. The nominal thickness shall be specified in inches or millimeters. Unless otherwise noted, the alloy shall be specified by its UNS designation.

Tolerances stricter than those listed in the relevant ASTM standards are permitted to be specified.

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: ASTM standards have more minimum order requirements for stainless steel than carbon other steel alloys because of the inherent differences between the steels and how they are used.

- Specification of the specific stainless steel alloy family and the specific alloy chemistry are critical because that determines 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 determines the strength level.
- The tighter flatness tolerance option is for applications where tight fit up or the appearance of flatness are critical. With a bare metal finish, variations in flatness are much more visible, particularly as finish reflectivity increases, and it may not be possible to completely correct the problem during fabrication.
- Finish specification is critical for corrosion performance and for applications with surface sanitation requirements. A rough finish, or one with mill scale or poor surface finish morphology can necessitate the
use of a higher alloyed stainless to avoid corrosion or more frequent than expected cleaning. Aesthetic appearance is dependent on corrosion performance and the consistency of surface finish morphology consistency. Please See the Commentary and AISC Design Guide 27 for more information.

User Note: ASTM standards does not define the minimum or nominal requirements for 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 carbon steel alloys and stainless steels are different. Additionally, the densities of each stainless steel alloy family and carbon steel alloys are slightly different.

Table User Note A3.1 lists some physical properties.

<table>
<thead>
<tr>
<th>Alloy Family</th>
<th>Modulus of Elasticity</th>
<th>Density</th>
<th>Thermal expansion</th>
<th>Thermal elongation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ksi</td>
<td>MPa</td>
<td>lb/ft³</td>
<td>kg/m³</td>
</tr>
<tr>
<td>Austenitic stainless steel</td>
<td>28,000</td>
<td>193,000</td>
<td>500</td>
<td>8,000</td>
</tr>
<tr>
<td>Duplex stainless steel</td>
<td>29,000</td>
<td>200,000</td>
<td>485</td>
<td>7,800</td>
</tr>
<tr>
<td>Precipitation hardening stainless steel</td>
<td>28,500</td>
<td>197,000</td>
<td>485</td>
<td>7,800</td>
</tr>
</tbody>
</table>

(a) Plate, sheet, strip (austenitic and duplex stainless steels only)
(1) Product form, ASTM A240/A240M, ASTM A480/A480M and the UNS designation shall be specified.
(2) The following requirements shall also be specified in accordance with ASTM A480/A480M:

Nominal dimensions including nominal thickness in inches or millimeters
Condition (hot-rolled, cold-rolled, annealed, or heat treated)
Standard or stretcher-leveled quality of flatness
Finish for the product form or a special finish agreed with the supplier:

Sheet: No. 1, No. 2, No. 2B, Bright Annealed (BA), No. 3, No. 4, No. 7, No. 8, TR;
Strip: No. 1, No. 2, Bright Annealed (BA), TR, polished;
Plate: Hot or cold rolled, and annealed or heat treated;
Hot- or cold-rolled, and annealed or heat treated, and blast cleaned or pickled;
Hot- or cold-rolled, and annealed or heat treated, and surface cleaned and polished;
Hot- or cold-rolled, and annealed or heat treated, and descaled, and temper passed;
Hot- or cold-rolled and annealed or heat treated and descaled and optionally temper passed.

**User Note:** Grinding or polishing of plate is a special finish requirement. There are numerous special finishing options for sheet that are not defined in ASTM A480.

**User Note:** The chemical and mechanical property requirements for each alloy are determined by ASTM A240/A240M. All general requirements are determined by ASTM A480/A480M including dimensional tolerances, flatness, finish, shipping and handling.

(b) Hollow structural sections (HSS) (austenitic and duplex stainless steels only)

(1) Mechanical (structural) welded HSS shall be specified using ASTM A554 and the UNS designation or the A554 Grade. Sharp cornered laser or laser hybrid welded tubing shall be specified using ASTM A1069/A1069M and the UNS designation or A554 Grade. The form (round, square, rectangular, special) and nominal dimensions shall be specified. For rounds, outside diameter and nominal thickness of the wall shall be specified.

**User Note:** For the cold-reduced condition, outside and inside diameter, or inside diameter and wall dimensions may be specified.

(2) One of the following conditions shall be specified:
- As welded,
- Welded and annealed,
- Cold reduced,
- Cold reduced and annealed.

(3) The following surface finish shall be specified:
- For ASTM A554 products, free of scale is the standard finish. If other conditions are required they shall be specified.
- For ASTM A1069 products, one of the following finishes shall be specified:
  - As-welded
  - Pickled in accordance with ASTM A380/A380M,
  - Descaled and passivated in accordance with ASTM A380/A380M.

**User Note:** Special finishes like polishing can be obtained but are typically applied after production by a polishing company.

**User Note:** Products manufactured in accordance with ASTM A554 and ASTM A1069/A1069M are available in round, square, rectangular and custom shapes. Pipe is only available in rounds. Dimensional tolerances are determined by ASTM A554 (ASTM A500 carbon steel tolerances shall not be used). ASTM A1069/A1069M square and rectangular HSS can have four sharp welded square corners while ASTM A554 product corners are rounded.
(c) Hollow structural pipe sections (austenitic and duplex stainless steels only)

(1) Pressure rated seamless or welded austenitic stainless steel pipe shall be specified using ASTM A312/A312M, ASTM A999/A999M, and the UNS designation. Pressure rated seamless or welded duplex stainless steel pipe shall be specified using ASTM A790/A790M, ASTM A999/A999M, and the UNS designation. Nominal dimensions and nominal thickness shall be specified.

(2) Seamless or welded pipe shall be specified.

User Note: Special finishes like polishing can be obtained but are typically applied by a polishing company rather than the product supplier.

User Note: Pressure rated pipe produced to ASTM A312/A312M and ASTM A790/A790M is for handling liquids and requires more testing than is typically required for structural applications which can increase cost, but, in larger sizes and heavier thicknesses, it maybe more readily available. Dimensional tolerances are determined by ASTM A999/A999M.

(d) Hot and cold finished bar, and flat bar cut from strip or plate

(1) ASTM A276/A276M or ASTM A479/A479M, ASTM A484/A484M, and the UNS designation shall be specified when ordering austenitic or duplex stainless steel round, square or hexagonal bar. ASTM A276/A276M, ASTM A484/A484M, and the UNS designation shall be specified when ordering austenitic or duplex stainless steel flat bar cut from plate. ASTM A564/A564M, ASTM A484/A484M, and the UNS designation shall be specified when ordering precipitation hardening stainless steel round, square or hexagonal bar. Nominal dimensions and nominal thickness shall be specified.

(2) One of the following ASTM A276/A276M conditions shall be specified to indicate the mechanical requirements:

- Condition A - annealed,
- Condition S - strain hardened with relatively light cold work,
- Condition B - relatively severe cold work.

(3) Finish for bar shall be specified in accordance with ASTM A484/A484M based on bar type. The standard finish options are given below but other finishes are also permitted:

Flattened 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

User Note: The bar used for most structural applications is typically dual certified to both ASTM A276/A276M and A479/A479M. The specifications are interchangeable for most structural applications. 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 (S3), and optimal resistance to chloride stress corrosion cracking (S5).

### User Note:
Dimensional tolerances are in accordance with ASTM A484/A484M.

### User Note:
For cold-finished bar in Condition B (relatively severe cold work) in accordance with ASTM A276/A276M, a reduced value for the modulus of elasticity $E_{red}$ should be used where $E_{red} = 25,000$ ksi (172 000 MPa).

#### (e) Extruded structural shapes

1. ASTM A276/A276M or ASTM A479/A479M, ASTM A484/A484M, and the UNS designation shall be specified when ordering austenitic or duplex stainless steel. ASTM A564/A564M, ASTM A484/A484M, and the UNS designation shall be specified when ordering precipitation hardening stainless steel. **Nominal dimensions and nominal thickness shall be specified.**

2. Shapes shall be descaled by machining, grinding, blasting, or pickling.

### User Note:
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. The standard finish is hot-finished. Dimensional tolerances are in accordance with ASTM A484/A484M.

#### (f) Hot-rolled shapes

1. ASTM A276/A276M, ASTM A484/A484M, and the UNS designation shall be specified when ordering austenitic or duplex stainless steel. ASTM A564/A564M, ASTM A484/A484M, and the UNS designation shall be specified when ordering precipitation hardening stainless steel. **Nominal dimensions and nominal thickness shall be specified.**

### User Note:
The standard finish is hot-finished. Special finishes like polishing can be obtained but are typically applied by a polishing company rather than the product supplier.

### User Note:
Currently only smaller hot rolled shapes are commonly available in stainless steel. The standard finish is hot-finished. Dimensional tolerances are in accordance with ASTM A484/A484M. **The hot-rolled structural shapes dimensional tolerances in ASTM A484/A484M are currently limited to hot finished equal and unequal leg angles (all sizes), channels up to 3 in. (75 mm), and tees up to 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 and do not cover as broad a size and shape range. The specifier can stipulate the ASTM A6/A6M Table 16, Table 17, or Table 18 shape tolerances, but acceptance is subject to contractual agreement between the purchaser and supplier. No part of ASTM A6/A6M other than dimensional tolerances is applicable since it is for other structural steels.**

#### (g) ASTM A1069/A1069M Laser welded and laser hybrid welded structural shapes

1. A1069/A1069M, ASTM A6 or custom dimensional tolerances, nominal
dimensions and nominal thickness, and the UNS designation shall be specified.

(2) Strength grade shall be specified in accordance with ASTM A1069/A1069M Table 1.

(3) The surface finish shall be specified as:

- As welded
- Pickled in accordance with ASTM A380/A380M.
- Descaled and passivated in accordance with ASTM A380/A380M.

User Note: Special finishes like polishing can be obtained but are typically applied by a polishing company rather than the product supplier.

(h) Welded Structural Sections Other than HSS and Laser or Laser Hybrid

(1) The appropriate plate or shape specifications from the preceding sections (a) to (f), and ASTM A6 tolerances, nominal dimensions and nominal thickness, and the UNS designation shall be specified.

(2) Suppliers of structural shapes, which are not welded by laser or laser-hybrid methods shall be required, as a minimum, to provide documentation or inspector certification of the following and adhere to the requirements of AWS D1.6/D1.6M:

a. Weld procedures shall be qualified in accordance with AWS B2.1/B2.1M. An AWS SWPS based on AWS B2.1/B2.1M (AWS B2.1-X-XXX series) is also acceptable. Prequalified AWS weld procedures for austenitic stainless steel or qualification of austenitic or duplex stainless steel weld procedures.

b. Current qualification of the welding personnel by testing for the welding procedures used.

c. Inspectors’ qualifications and responsibilities.

d. Filler metal, if used.

e. Certify that surfaces are cleaned prior to and after welding. Chemical descaling and other weld corrosion resistance restoration methods shall be in accordance with ASTM A380/A380M, removing all mill scale; weld splatter and other weld defects that could adversely affect corrosion performance.

User Note: AWS D1.6/D1.6M specifically covers welded structures. It does not provide requirements for welding of the structural shapes that form those structures. Where ASTM product specifications or AWS D1.6/D1.6M does not provide requirements for tolerances of joint dimensions, AWS D1.1/D1.1M clause 5.22 will be used. ASTM has product specifications for ASTM A1069/A1069M and the HSS welded structural sections but does not currently have specifications for structural shapes welded by other means.

User Note: When corrosion testing of duplex stainless steel welds is required, Standard Test Method ASTM A1084 is used for the lean duplex-lower alloyed steels UNS S32101, S32202, and S32304, and S32003, and Standard Test Methods ASTM A923 is used for the higher alloyed duplex steels S32003, S32205, S32760, S32750, and S82441.
(i) **Welded assemblies**

In addition to specification of the relevant sections above, adherence with the requirements of AWS D1.6/D1.6M shall be required for welded assemblies. Also see Section J2.

**User Note:** The user note on corrosion testing of duplex stainless steels in Section A3.1b (h) also applies.

**User Note:** Table User Notes A3.2 to A3.7 gives specified minimum mechanical properties for the products covered in sections (a) to (g) above according to the relevant ASTM specifications/standards.

The User Note on corrosion testing of duplex stainless steels in Section A3.1b (h) also applies.

**User Note:** The ‘L’ in some austenitic stainless steel designations indicates a low carbon version. When producers use state-of-the-art production methods, stainless steels are often low carbon and dual certified to both designations (e.g., S30400/ S30403, with the higher strength of S30400 and the lower carbon content of S30403, and S31600/S31603, with the higher strength of S31600 and the lower carbon content of S31603).
### Table User Note A3.2

**Minimum Mechanical Property Requirements for Plate, Sheet, and Strip (ASTM A240/A240M-18)**

<table>
<thead>
<tr>
<th>UNS Designation</th>
<th>Type</th>
<th>Tensile Strength, ( F_u )</th>
<th>Yield Strength, ( F_y )</th>
<th>Elongation in 2 in. or 50 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>ksi</td>
<td>MPa</td>
<td>ksi</td>
</tr>
<tr>
<td><strong>Austenitic</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>N08367:</td>
<td>Sheet/strip</td>
<td>–</td>
<td>100</td>
<td>690</td>
</tr>
<tr>
<td></td>
<td>Plate</td>
<td></td>
<td>95</td>
<td>655</td>
</tr>
<tr>
<td>N08904</td>
<td></td>
<td>904L</td>
<td>71</td>
<td>490</td>
</tr>
<tr>
<td>N08926</td>
<td></td>
<td>–</td>
<td>94</td>
<td>650</td>
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<tr>
<td>S30400</td>
<td></td>
<td>304</td>
<td>75</td>
<td>515</td>
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<td>S30403</td>
<td></td>
<td>304L</td>
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<td>S31600</td>
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<td>515</td>
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<tr>
<td>S32100</td>
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<td>321</td>
<td>75</td>
<td>515</td>
</tr>
<tr>
<td>S31254:</td>
<td>Sheet/strip</td>
<td>–</td>
<td>100</td>
<td>690</td>
</tr>
<tr>
<td></td>
<td>Plate</td>
<td></td>
<td>95</td>
<td>655</td>
</tr>
<tr>
<td><strong>Duplex (Austenitic-Ferritic)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S32003:</td>
<td></td>
<td>–</td>
<td>95</td>
<td>655</td>
</tr>
<tr>
<td>t&gt;0.187 in [5.0 mm]</td>
<td></td>
<td>–</td>
<td>94</td>
<td>650</td>
</tr>
<tr>
<td>S32101</td>
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<td>–</td>
<td>94</td>
<td>650</td>
</tr>
<tr>
<td>S32202</td>
<td></td>
<td>–</td>
<td>94</td>
<td>650</td>
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<tr>
<td>S32205</td>
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<td>2205</td>
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<td>655</td>
</tr>
<tr>
<td>S32304</td>
<td></td>
<td>2304</td>
<td>87</td>
<td>600</td>
</tr>
<tr>
<td>S32750</td>
<td></td>
<td>2507</td>
<td>116</td>
<td>795</td>
</tr>
<tr>
<td>S32760</td>
<td></td>
<td>–</td>
<td>108</td>
<td>750</td>
</tr>
<tr>
<td>S82011:</td>
<td></td>
<td>–</td>
<td>95</td>
<td>655</td>
</tr>
<tr>
<td>t&gt;0.187 in [5.0 mm]</td>
<td></td>
<td>–</td>
<td>95</td>
<td>655</td>
</tr>
<tr>
<td>S82441:</td>
<td></td>
<td>–</td>
<td>107</td>
<td>740</td>
</tr>
<tr>
<td>t&lt;0.4 in [10.0 mm]</td>
<td></td>
<td>–</td>
<td>99</td>
<td>680</td>
</tr>
<tr>
<td>t&gt;0.4 in [10.0 mm]</td>
<td></td>
<td>–</td>
<td>79</td>
<td>580</td>
</tr>
</tbody>
</table>

*Only thicknesses above 0.187 in. (5 mm) are shown.*

**User Note:** The ‘L’ in some austenitic stainless steel designation indicates a low carbon version. When producers use state of the art production methods, commercially produced stainless steels are often low carbon and dual certified to both designations (e.g., S30400/S30403, with the higher strength of S30400 and the lower carbon content of S30403, and S31600/S31603, with the higher strength of S31600 and the lower carbon content of S31603). Only thicknesses above 0.187 in. (5 mm) are shown.
## Table User Note A3.3
Minimum Mechanical Property Requirements for Welded Mechanical Tubing (ASTM A554/A554M-16)

<table>
<thead>
<tr>
<th>UNS Designation</th>
<th>Type</th>
<th>Tensile Strength, ( F_u )</th>
<th>Yield Strength, ( F_y )</th>
<th>Elongation in 2 in. or 50 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>ksi</td>
<td>MPa</td>
<td>ksi</td>
</tr>
<tr>
<td><strong>Austenitic stainless steels</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S30403 &amp; S31603</td>
<td>MT 304L</td>
<td>70</td>
<td>483</td>
<td>25</td>
</tr>
<tr>
<td>All other austenitic stainless steels</td>
<td>–</td>
<td>75</td>
<td>517</td>
<td>30</td>
</tr>
<tr>
<td><strong>Duplex stainless steels</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S32003: t≤0.187 in [5.0 mm]</td>
<td>–</td>
<td>100</td>
<td>690</td>
<td>70</td>
</tr>
<tr>
<td>t&gt;0.187 in [5.0 mm]</td>
<td>95</td>
<td>655</td>
<td>65</td>
<td>450</td>
</tr>
<tr>
<td>S32101: t&lt;0.4 in [10.0 mm]</td>
<td>–</td>
<td>101</td>
<td>700</td>
<td>77</td>
</tr>
<tr>
<td>t≥0.4 in [10.0 mm]</td>
<td>94</td>
<td>650</td>
<td>65</td>
<td>450</td>
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<tr>
<td>S32202</td>
<td>94</td>
<td>650</td>
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</tr>
<tr>
<td>S32205</td>
<td>2205</td>
<td>95</td>
<td>655</td>
<td>65</td>
</tr>
<tr>
<td>S32304</td>
<td>2304</td>
<td>87</td>
<td>600</td>
<td>58</td>
</tr>
<tr>
<td>S32750</td>
<td>2507</td>
<td>116</td>
<td>795</td>
<td>80</td>
</tr>
<tr>
<td>S32760</td>
<td>–</td>
<td>108</td>
<td>750</td>
<td>80</td>
</tr>
<tr>
<td>S82011: t≤0.187 in [5.0 mm]</td>
<td>–</td>
<td>101</td>
<td>700</td>
<td>75</td>
</tr>
<tr>
<td>t&gt;0.187 in [5.0 mm]</td>
<td>95</td>
<td>655</td>
<td>65</td>
<td>450</td>
</tr>
<tr>
<td>S82441: t&lt;0.4 in [10.0 mm]</td>
<td>–</td>
<td>107</td>
<td>740</td>
<td>78</td>
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<tr>
<td>t≥0.4 in [10.0 mm]</td>
<td>99</td>
<td>680</td>
<td>70</td>
<td>480</td>
</tr>
</tbody>
</table>
## Table User Note A3.4

Minimum Mechanical Property Requirements for Seamless or Welded Duplex Stainless Steel Pipe (ASTM A790/A790M-19)

<table>
<thead>
<tr>
<th>UNS Designation</th>
<th>Type</th>
<th>Tensile Strength, $F_u$</th>
<th>Yield Strength, $F_y$</th>
<th>Elong. in 2 in. or 50 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>ksi</td>
<td>MPa</td>
<td>ksi</td>
</tr>
<tr>
<td>S32003: t≤0.187 in [5.0 mm]</td>
<td>–</td>
<td>100</td>
<td>690</td>
<td>70</td>
</tr>
<tr>
<td>S32003: t&gt;0.187 in [5.0 mm]</td>
<td>95</td>
<td>655</td>
<td>65</td>
<td>450</td>
</tr>
<tr>
<td>S32101: t≤0.187 in [5.0 mm]</td>
<td>101</td>
<td>700</td>
<td>77</td>
<td>530</td>
</tr>
<tr>
<td>S32101: t&gt;0.187 in [5.0 mm]</td>
<td>94</td>
<td>650</td>
<td>65</td>
<td>450</td>
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<td>S32202</td>
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<td>80</td>
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<tr>
<td>S32760</td>
<td>–</td>
<td>109</td>
<td>750</td>
<td>80</td>
</tr>
<tr>
<td>S82011: t≤0.187 in [5.0 mm]</td>
<td>–</td>
<td>95</td>
<td>655</td>
<td>65</td>
</tr>
<tr>
<td>S82011: t&gt;0.187 in [5.0 mm]</td>
<td>101</td>
<td>700</td>
<td>75</td>
<td>515</td>
</tr>
<tr>
<td>S82441: Wall &lt;0.4 in. [10.0 mm]</td>
<td>–</td>
<td>107</td>
<td>740</td>
<td>78</td>
</tr>
<tr>
<td>S82441: Wall ≥0.4 in. [10.0 mm]</td>
<td>99</td>
<td>680</td>
<td>70</td>
<td>480</td>
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Table User Note A3.5
Minimum Mechanical Property Requirements for Bars and Shapes
(ASTM A276/A276M-17)

<table>
<thead>
<tr>
<th>Designation</th>
<th>Condition (A, S or B), finish and size [in. (mm)]</th>
<th>Tensile Strength, (F_u) ksi</th>
<th>Yield Strength, (F_y) ksi</th>
<th>Elong. in 2 in. or (50 mm) %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>ksi</td>
<td>MPa</td>
<td></td>
</tr>
<tr>
<td>Austenitic</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>N08367</td>
<td>A, HF or CF</td>
<td>95</td>
<td>655</td>
<td>45</td>
</tr>
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<td>N08904 904L</td>
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<td>N08926</td>
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<td>304, 316</td>
<td></td>
<td>75</td>
<td>515</td>
<td>30</td>
</tr>
<tr>
<td>304, 316</td>
<td>≤ 0.5 (12.70)</td>
<td>90</td>
<td>620</td>
<td>45</td>
</tr>
<tr>
<td></td>
<td>&gt; 0.5 (12.70)</td>
<td>75</td>
<td>515</td>
<td>30</td>
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<tr>
<td>304L, 316L</td>
<td>A, HF</td>
<td>70</td>
<td>485</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td>≤ 0.5 (12.70)</td>
<td>90</td>
<td>620</td>
<td>45</td>
</tr>
<tr>
<td></td>
<td>&gt; 0.5 (12.70)</td>
<td>70</td>
<td>485</td>
<td>25</td>
</tr>
<tr>
<td>304L, 316L</td>
<td>S, CF:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 1.5 (38.10)</td>
<td>95</td>
<td>655</td>
<td>45</td>
</tr>
<tr>
<td></td>
<td>≤ 1.75 (44.45)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 1.75 (44.45)</td>
<td>95</td>
<td>650</td>
<td>75</td>
</tr>
<tr>
<td></td>
<td>≤ 2.0 (50.8)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 2.0 (50.8)</td>
<td>90</td>
<td>620</td>
<td>65</td>
</tr>
<tr>
<td></td>
<td>≤ 2.5 (63.5)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 2.5 (63.5)</td>
<td>80</td>
<td>550</td>
<td>55</td>
</tr>
<tr>
<td>304, 304L,</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>316, 316L</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>≤ 3/4 (19.05)</td>
<td>125</td>
<td>860</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>&gt; 0.75 (19.05)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>≤ 1.25 (31.75)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 1.25 (31.75)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>≤ 1.5 (38.10)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Duplex (Austenitic-Ferritic)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S32101</td>
<td>A</td>
<td>94</td>
<td>650</td>
<td>65</td>
</tr>
<tr>
<td>S32202</td>
<td>A</td>
<td>94</td>
<td>650</td>
<td>65</td>
</tr>
<tr>
<td>S32205</td>
<td>A</td>
<td>95</td>
<td>655</td>
<td>65</td>
</tr>
<tr>
<td>S32304</td>
<td>A</td>
<td>87</td>
<td>600</td>
<td>58</td>
</tr>
<tr>
<td>S32750</td>
<td>A: ≤ 2.0 (50.8)</td>
<td>116</td>
<td>800</td>
<td>80</td>
</tr>
<tr>
<td></td>
<td>A: &gt; 2.0 (50.8)</td>
<td>110</td>
<td>760</td>
<td>75</td>
</tr>
<tr>
<td>S32760</td>
<td>A</td>
<td>109</td>
<td>750</td>
<td>80</td>
</tr>
<tr>
<td>S32760</td>
<td>S</td>
<td>125</td>
<td>860</td>
<td>105</td>
</tr>
<tr>
<td>S82441</td>
<td>A: &lt; 7/16 (11)</td>
<td>107</td>
<td>740</td>
<td>78</td>
</tr>
<tr>
<td></td>
<td>A: ≥ 7/16 (11)</td>
<td>99</td>
<td>680</td>
<td>70</td>
</tr>
</tbody>
</table>

A = Annealed, S = Strain Hardened (Relatively light cold work), B = Relatively severe cold work, HF = hot finished, CF = cold finished

User Note: Alloys such as 304 and 304L are often dual certified to the higher of the two minimum requirement levels.
Table User Note A3.6
Minimum Mechanical Property Requirements for Precipitation Hardening Stainless Steels Bars and Shapes (ASTM A564/A564M-19a)

<table>
<thead>
<tr>
<th>UNS (Common Name)</th>
<th>Heat Treatment Condition</th>
<th>Tensile Strength, $F_u$ (ksi)</th>
<th>Yield Strength, $F_y$ (ksi)</th>
<th>Elongation in 2 in. (50mm) or 4D (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S15500 (XM-12)</td>
<td>H1150</td>
<td>135</td>
<td>105</td>
<td>16</td>
</tr>
<tr>
<td></td>
<td>H1150M</td>
<td>115</td>
<td>75</td>
<td>18</td>
</tr>
<tr>
<td>S17400 (630)</td>
<td>H1150</td>
<td>135</td>
<td>105</td>
<td>16</td>
</tr>
<tr>
<td></td>
<td>H1150M</td>
<td>115</td>
<td>75</td>
<td>18</td>
</tr>
<tr>
<td></td>
<td>H1150D</td>
<td>125</td>
<td>105</td>
<td>16</td>
</tr>
</tbody>
</table>

**User Note:** Precipitation hardening stainless steels in conditions other than H1150 present a significant risk of hydrogen embrittlement and should not be used for structurally critical applications. Furthermore, precipitation hardening stainless steels are the least corrosion resistant of the stainless steels in this specification. They are not suitable for most exterior or for any swimming pool building applications unless completely encased and protected from the environment.
### Minimum Mechanical Property Requirements for Laser and Laser-Hybrid Welded Bars, Plates and Shapes (A1069/A1069M)

<table>
<thead>
<tr>
<th>Strength Grade</th>
<th>Tensile Strength, $F_u$</th>
<th>Yield Strength, $F_y$</th>
<th>Elongation in 2 in. or 50 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ksi</td>
<td>MPa</td>
<td>ksi</td>
</tr>
<tr>
<td>1</td>
<td>Refer to the minimum mechanical property requirements in ASTM A240/A240M, A276, A554 or A479/A479M</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Austenitic</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>80</td>
<td>550</td>
<td>35</td>
</tr>
<tr>
<td>3**</td>
<td>95</td>
<td>655</td>
<td>65</td>
</tr>
<tr>
<td>4***</td>
<td>116</td>
<td>795</td>
<td>80</td>
</tr>
</tbody>
</table>

**Notes:**
- * This strength level applies to the following austenitics: UNS designation (common name): UNS S30403 (304L), S30409 (304HL), S31603 (316L), S31653 (316LN), and S31703 (317L). Order all other austenitics should be ordered to strength Grade 1, ASTM A240/A240M mechanical properties.
- ** This strength level applies to S32205 (2205) up to 2.5 in. (64 mm) in thickness.
- *** This strength level can be achieved by applying to the more highly alloyed, more corrosion resistant and higher strength super duplexes like S32750, and S32760, and is limited to thicknesses of up to 2 in. (50 mm) in thickness.

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### 1c. Minimum Specification Requirements for Corrosion Resistance to Avoid Corrosion Failure

Minimum alloy specification requirements for corrosive service environments are established in this section.

#### 1. Galvanic Corrosion

When dissimilar metals will be 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. If dissimilar metals will be in direct contact in any location, the design shall be assessed to determine the potential for galvanic corrosion. The EOR shall specify how the metals are to be isolated, if required. This shall include adequate separation of all dissimilar metal sections using inert materials, including coatings applied to welded joints, and sealing bimetallic mechanically fastened joints.

**User Note:** See Commentary, AISC Design Guide 27, and Sections J2.8, J3.13, and M2. Stainless steel is typically the most noble of the metals being joined and the other metal(s) 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 galvanic potential, relative surface area of the bare metals that are in direct contact, presence of moisture, including humidity, fog, condensation, rain, and immersion on a regular basis, and the presence of pollution and chloride salts that can
accelerate galvanic corrosion. ASTM G71 or G82 testing may be used to help define the magnitude of the problem. Appropriate metal separation methods can vary with the service environment. Water shedding atmospheric and immersed applications typically have different separation requirements. See AISC Design Guide 27, and Sections J2.8 Welding Dissimilar Metals, J3.13 Bolting Dissimilar Metals, and M2 Fabrication.

2. Chloride Stress Corrosion Cracking

Structural members with exposure to chloramine or chloride salts shall be assessed for their potential to chloride stress corrosion cracking. For service environments where chloride stress corrosion cracking of structural members is a concern, they shall be made from duplex S32205 or an alloy(s) with greater resistance to chloride stress corrosion cracking unless a stainless steel corrosion expert determines that a less corrosion resistant alloy is acceptable. In applications that are highly loaded, particularly when there are heavily cold worked stainless steel components with high levels of chloramine or chloride salt exposure, the alloy specified shall have adequate chloride stress corrosion cracking resistance for the service environment.

The EOR shall at minimum specify duplex S32205 but a 6% molybdenum or higher austenitic stainless steel or super duplex is often necessary where there is a significant risk of chloride stress corrosion cracking, such as:

- elevated areas in indoor swimming pools
- elevated sections of glass curtain walls adjoining indoor swimming pools
- certain industrial and food processing environments
- the interface between air and brackish water or seawater
- areas with high coastal or deicing salts accumulation

User Note: Common austenitic stainless steels (i.e., 340/304L, 316/316L) 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.

User Note: Refer to the Commentary of Section A2 for guidance on corrosion assessment and AISC Design Guide 27 for more information.

3. Crevice Corrosion

Connections shall be assessed for their crevice corrosion potential. A suitable stainless steel alloy(s) shall be selected for the service environment. The EOR shall determine whether tight crevices are expected as a result of mechanical fastening, incomplete welding, surface deposits produced by microbiological organisms, or other surface deposit accumulations during use, and select a stainless steel alloy(s) with suitable crevice corrosion resistance for the service environment.

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Public Review draft dated October 14, 2020
AMERICAN INSTITUTE OF STEEL CONSTRUCTION
4. Salt or Brackish Water Immersion

For applications that will be immersed in brackish water or seawater, the EOR shall at minimum specify S32205 or alloys of equivalent or better corrosion resistance unless a stainless steel corrosion expert determines that a less corrosion resistant alloy is acceptable.

For brackish water or seawater applications with microbiological organisms, chlorination, high aeration, pollution, other chemical additions or other factors that increase water corrosivity, the EOR shall at minimum specify a 6% molybdenum austenitic stainless steel or a super duplex seawater stainless steel unless a stainless steel corrosion expert determines that a less corrosion resistant alloy is acceptable.

1d. 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 primary (computed) tensile forces due to tension or flexure and spliced or connected to other members using complete-joint-penetration groove welds that fuse through the thickness of the plates, shall be specified as follows. The structural design documents shall require that the steel be supplied with Charpy V-notch impact test results 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: This requirement does not apply to austenitic stainless steels because they are not susceptible to brittle fracture, even at low temperatures. They demonstrate impact toughness well above 74 ft-lbf (100 J) at −320°F (−196°C).

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

2. 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
standard specifications or standard test methods shall constitute sufficient evidence of conformity with such standards. The following specifications apply:

Castings
ASTM A351 (austenitic stainless steel)
ASTM A890 (duplex stainless steel)
ASTM A747 (precipitation hardening stainless steel)

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

User Note: ASTM standards specifications do not cover casting or forging design, nor are they manufacturing processes. Mechanical properties, weldability, non-destructive examination, and other requirements should be considered to achieve the level of service expected for the application.

3. Bolts, Washers, and Nuts

Bolts, washers, and nuts material made of austenitic, duplex or precipitation hardening stainless steel conforming to one of the following ASTM standards specifications, depending on the corrosion resistance requirements, is approved for use under this Specification:

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

(b) Nuts
ASTM A194/A194M
ASTM A962/A962M
ASTM F594
ASTM F856M

(c) Washers
The chemical composition and mechanical properties of the washer raw material shall meet the requirements of ASTM A240/A240M, A666/A666M, or A693/A693M. Unless otherwise specified, the requirements for dimensions, tolerances and hardness testing of ASTM F436 shall apply.

User Note: There are no ASTM standards for stainless steel washers. Purchasers shall specify that the washer raw material has a chemical composition and mechanical properties that meet the requirements of ASTM A240/A240M, A666/A666M, or A693/A693M. Unless otherwise specified, the requirements for dimensional tolerances and hardness testing of ASTM F436 shall apply.
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.

Nonferrous nickel alloy bolts shall conform to one of the following ASTM standards, depending on the corrosion resistance requirements, are approved for use under this Specification:

- ASTM F468
- ASTM F468M

**User Note:** Nickel alloy bolts are commonly used for very corrosive applications and can potentially provide higher strength levels than duplex bolts. The difference in their surface hardness and that of stainless steels is also utilized to prevent galling.

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

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

- 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 B18.2.6 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 structural stainless steel are permitted at a temperature below 850°F (450°C):

- 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, so this approach is discouraged. If this method is used, the austenitic stainless steel alloy must 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.

- Low temperature carburizing of duplex stainless steel
- 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 metals shall conform to the requirements of AWS D1.6/D1.6M. Filler and base metal combinations shall be in accordance with the prequalified materials listed in AWS D1.6/D1.6M clause 5, one of the options suggested in AWS D1.6/D1.6M Annex D, 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 metal combinations that do not meet the prequalification requirements of AWS D1.6/D1.6M clause 5 conform to the specifications of the American Welding Society.

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

A4. 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 A3.1d: 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, handling 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:

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 Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE/SEI 7).

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

B3. DESIGN BASIS

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

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

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

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

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

\[ R_u \leq \phi R_n \]  

(B3-1)

where

- \( R_u \) = required strength using LRFD load combinations
- \( R_n \) = nominal strength
- \( \phi \) = resistance factor
- \( \phi R_n \) = design strength

The nominal strength, \( R_n \), and the resistance factor, \( \phi \), 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:

\[ R_e \leq \frac{R_n}{\Omega} \]  

(B3-2)

where

- \( R_e \) = required strength using ASD load combinations
- \( R_n \) = nominal strength
- \( \Omega \) = safety factor
- \( R_n / \Omega \) = allowable strength

The nominal strength, \( R_n \), and the safety factor, \( \Omega \), 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: Section 3.1.2 of the Code of Standard Practice for Structural Stainless Steel Buildings 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

$$D = \text{nominal dead load, kips (N)}$$

$$L = \text{nominal live load, kips (N)}$$

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

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

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

10. Design for Ponding

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

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

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

11. Design for Fatigue

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

12. Design for Fire Conditions

Two methods of design for fire conditions are provided in Appendix 4: (a) by analysis and (b) by qualification testing.
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: Hot rolled, extruded and laser or laser-hybrid welded sections have joint strengths equivalent to the base metal. If the production route for the section is not known at the time of design, the section may be conservatively treated as a built-up section when computing its cross-sectional properties required for the calculation of strengths in accordance with Chapters E through K and Appendices 2 and 4.

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 \( \lambda_r \) from Table B4.1a. If the width-to-thickness ratio of any compression element exceeds \( \lambda_r \), 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, \( \lambda_p \), from Table B4.1b. If the width-to-thickness ratio of one or more compression elements exceeds \( \lambda_p \), but does not exceed \( \lambda_r \) from Table B4.1b, the section is noncompact. If the width-to-thickness ratio of any compression element exceeds \( \lambda_r \), 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) which are designed on the basis of the average yield strength, \( F_{y,avg} \), as prescribed in Section B4.32, 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.

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; $h_{pl}$ 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 Wall Thickness for HSS

The design wall thickness, $t$, shall be used in calculations involving the wall thickness of hollow structural sections (HSS) specified in A3.1(b) and (c). The design wall thickness, $t$, shall be taken equal to 0.93 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.
2b. Design Thickness for Built-Up Sections

The design thickness, $t_f$ for the flanges, and $t_w$ for the webs, shall be used in calculations involving the thickness of built-up sections fabricated from the products specified in Sections A3.1b (a), (d), and (h). The design thickness, $t_f$ or $t_w$, shall be taken equal to the nominal thickness for thicknesses greater than or equal to 3/16 in. (5 mm). For thicknesses less than 3/16 in. (5 mm), the design thickness, $t_f$ or $t_w$, shall be taken equal to 0.95 times the nominal thickness.

2c. Design Thickness for Hot-Rolled Sections

The design thickness, $t_f$ for the flanges, and $t_w$ for the webs, shall be used in calculations involving the thickness of hot-rolled sections specified in Section A3.1b(f). The design thickness, $t_f$ or $t_w$, shall be taken equal to the nominal thickness for sections of nominal size greater than or equal to 3 in. (75 mm). For sections of nominal size smaller than 3 in. (75 mm), the design thickness, $t_f$ or $t_w$, shall be taken equal to 0.95 times the nominal thickness.

32. Strength Increase in Stainless Steel HSS from Cold Forming

Strength increase from cold work in structural 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 structural 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,
2. Stub column tests in accordance with AISI S902,
3. Computed in accordance with Equation B4-1:

\[ F_{y,avg} = F_{y,corner} + F_{y,wall} \left( \frac{A_g - A_{corner}}{A_g} \right) \leq F_u \]  
(B4-1a)

(b) For cold-formed round HSS

\[ F_{y,avg} = F_{y,round} \]  
(B4-1b)

where

\[ A_g = \text{gross area of member, in.}^2 \text{ (mm}^2\text{)} \]
\[ A_{corner} = \text{total corner area including a region of length } 2t \text{ extending around the perimeter of the cross section on both sides of each corner, in.}^2 \text{ (mm}^2\text{)} \]
\[ = \left( \pi t \right) \left( 2t + t \right) + 16 t^2 \]  
(B4-2)

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

\[ F_{y,corner} = 0.85 \frac{F_y}{\varepsilon_y} \left( \varepsilon_{corner} + \varepsilon_y \right) \text{ and } F_y \leq F_{y,corner} \leq F_u \]  
(B4-3)

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

\[ F_{y,round} = 0.85 \frac{F_y}{\varepsilon_y} \left( \varepsilon_{wall} + \varepsilon_y \right) \text{ and } F_y \leq F_{y,round} \leq F_u \]  
(B4-4)

$F_{y,round}$ = tensile yield stress of cold-formed round HSS, ksi (MPa)
\[ \varepsilon_{\text{corner}} = \frac{0.85 F_y}{E \varepsilon_y} (\varepsilon_{\text{rnd}} + \varepsilon_y)^n \quad \text{and} \quad F_y \leq F_{y,\text{rnd}} \leq F_u \quad (B4-5) \]

\[ \varepsilon_{\text{corner}} = \text{strain induced in the corner of cold-formed rectangular HSS} \]

\[ \varepsilon_{\text{wall}} = \frac{t}{2(2\eta + t)} \quad (B4-6) \]

\[ \varepsilon_{\text{wall}} = \text{strain induced in the flats of cold-formed rectangular HSS} \]

\[ \varepsilon_{\text{wall}} = \frac{t}{35.43} \left( \frac{\pi t}{2(b + h - 2t)} \right) \quad \text{if } t, b, \text{ and } h \text{ are given in in.} \quad (B4-7a) \]

\[ \varepsilon_{\text{wall}} = \frac{t}{900} + \frac{\pi t}{2(b + h - 2t)} \quad \text{if } t, b, \text{ and } h \text{ are given in mm} \quad (B4-7b) \]

\[ \varepsilon_{\text{wall}} = \text{strain induced in cold-formed round HSS} \]

\[ \varepsilon_{\text{wall}} = \frac{t}{2(2\eta + t)} \quad (B4-8) \]

\[ \varepsilon_y = 0.002 + \frac{F_y}{E} \quad (B4-9) \]

\[ n = \log \left( \frac{F_y}{F_u} \right) \quad (B4-10) \]

\[ r_i = \text{internal corner radii, which may be taken as } 2t \text{ if not known, in. (mm)} \]

\[ E = \text{modulus of elasticity of stainless steel} \]

\[ = 28,000 \text{ ksi (193 000 MPa)} \text{ for austenitic, and 29,000 ksi (200 000 MPa) for duplex stainless steel} \]

\[ F_u = \text{specified minimum tensile strength, ksi (MPa)} \]

\[ F_y = \text{specified minimum yield stress, ksi (MPa)} \]

\[ \varepsilon_u = \text{ultimate strain, which may be approximated according to Equation (B4-2) if not known} \]

\[ \varepsilon_f = \text{specified minimum elongation after fracture determined over a length of 2 in. (50 mm)} \quad (B4-112) \]

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

43. Gross and Net Area Determination

43a. Gross Area

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

43b. 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 $s^2/4g^2$,

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, fabrication, and erection 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**

<table>
<thead>
<tr>
<th>Case</th>
<th>Description of Element</th>
<th>Width-to-Thickness Ratio</th>
<th>Limiting Width-to-Thickness Ratio $\lambda_r$ (nonslender / slender)</th>
<th>Examples</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>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, and flanges and stems of tees</td>
<td>$b/t$</td>
<td>$0.417 \frac{E}{F_y}$</td>
<td>![Flanges Diagram]</td>
</tr>
<tr>
<td>2</td>
<td>Legs of single angles, legs of double angles with separators, and all other unstiffened elements</td>
<td>$b/t$</td>
<td>$0.38 \frac{E}{F_y}$</td>
<td>![Legs Diagram]</td>
</tr>
<tr>
<td>3</td>
<td>Webs of built-up or rolled I-shaped sections and channels, and walls of rectangular HSS and box sections of uniform thickness</td>
<td>$ht_{tw}$</td>
<td>$1.24 \frac{E}{F_y}$</td>
<td>![Webs Diagram]</td>
</tr>
<tr>
<td>4</td>
<td>Round HSS</td>
<td>$D/t$</td>
<td>$0.10 \frac{E}{F_y}$</td>
<td>![Round HSS Diagram]</td>
</tr>
</tbody>
</table>

- $E =$ 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 or the Table User Notes in Section A3.
**TABLE B4.1b**

**Width-to-Thickness Ratios: Compression Elements Members Subject to Flexure**

<table>
<thead>
<tr>
<th>Case</th>
<th>Description of Element</th>
<th>Width-to-Thickness Ratio</th>
<th>Limiting Width-to-Thickness Ratios</th>
<th>Examples</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>Flanges of built-up or rolled I-shaped sections and channels in flexure about the major or minor axis</td>
<td>$b/t$</td>
<td>$0.33 \sqrt[3]{\frac{E}{F_y}}$</td>
<td>$0.417 \sqrt[3]{\frac{E}{F_y}}$</td>
</tr>
<tr>
<td>6</td>
<td>Webs of built-up or rolled I-shaped sections, channels, rectangular HSS and box sections of uniform thickness</td>
<td>$h/t$ or $b/t$</td>
<td>$2.54 \sqrt[3]{\frac{E}{F_y}}$</td>
<td>$3.01 \sqrt[3]{\frac{E}{F_y}}$</td>
</tr>
<tr>
<td>7</td>
<td>Flanges of rectangular HSS and box sections of uniform thickness</td>
<td>$b/t$</td>
<td>$1.17 \sqrt[3]{\frac{E}{F_y}}$</td>
<td>$1.24 \sqrt[3]{\frac{E}{F_y}}$</td>
</tr>
<tr>
<td>8</td>
<td>Round HSS</td>
<td>$D/t$</td>
<td>$0.08 \sqrt[3]{\frac{E}{F_y}}$</td>
<td>$0.31 \sqrt[3]{\frac{E}{F_y}}$</td>
</tr>
</tbody>
</table>

*Note: $E$ = modulus of elasticity of stainless steel, ksi (MPa), $F_y$ = specified minimum yield stress, ksi (MPa), given in the relevant ASTM standard or the Table User Notes in Section A5.*
CHAPTER C

DESIGN FOR STABILITY

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

The chapter is organized as follows:

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

User Note: Alternative methods for the design of structures for stability are provided in 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 $P-\Delta$ and $P-\delta$ effects); (c) geometric imperfections; (d) stiffness reductions due to inelasticity, including the effect of partial yielding of the cross section which may be accentuated by the presence of residual stresses; and (e) uncertainty in system, member, and connection strength and stiffness. All load-dependent effects shall be calculated at a level of loading corresponding to LRFD load combinations or 1.6 times ASD load combinations.

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

User Note: See Commentary Section C1 and Table C-C1.1 for an explanation of how requirements (a) through (e) of Section C1 are satisfied.

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 $P$-$\Delta$ and $P$-$\delta$ effects, except that it is permissible to neglect the effect of $P$-$\delta$ 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 $P$-$\delta$ 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 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 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:

\[ N_i = 0.002 \alpha Y_i \]  

(C2-1)

where

\[ \alpha = 1.0 \text{ (LRFD)}; \quad \alpha = 1.6 \text{ (ASD)} \]

\[ N_i = \text{notional load applied at level } i, \text{ kips (N)} \]

\[ Y_i = \text{gravity load applied at level } i \text{ from the LRFD load combination or ASD load combination, as applicable, kips (N)} \]
User Note: The use of notional loads can lead to additional (generally small) fictitious base shears in the structure. The correct horizontal reactions at the foundation may be obtained by applying an additional horizontal force at the base of the structure, equal and opposite in direction to the sum of all notional loads, distributed among vertical load-carrying elements in the same proportion as the gravity load supported by those elements. The notional loads can also lead to additional overturning effects, which are not fictitious.

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

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

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

User Note: An out-of-plumbness of 1/500 represents the maximum tolerance on column plumbness specified in the *Code of Standard Practice 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, \( \tau \), given by Table C23.1 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, $\tau_b$, shall be applied to the flexural stiffnesses of all members whose flexural stiffnesses are considered to contribute to the stability of the structure.

$$\tau_b = \frac{1.0}{1.00 + 0.002 \left( \frac{E}{\alpha P_r / A} \right)^{n_{eff}} / P_{ns}} \quad (C2-2)$$

where

- $\alpha_r = 1.0$ (LRFD); $\alpha_r = 1.6$ (ASD)
- $P_r = \text{required axial compressive strength using LRFD or ASD load combinations, kips (N)}$
- $P_{ns} = \text{cross-section compressive strength; for nonslender-element sections, } P_{ns} = F_{y} A_{e}$, and for slender-element sections, $P_{ns} = F_{y} A_{e}$, where $A_{e}$ is as defined in Section E7, kips (N)
- $n_{eff} = \text{auxiliary coefficient given in Table C23.1, where } n \text{ is the strain hardening coefficient as given in Appendix 7.}$

User Note: Taken together, Sections (a) and (b) require the use of $\tau_{g} \tau_{b}$ times the nominal elastic flexural stiffness and $\tau_{g}$ times other nominal elastic stiffnesses for structural steel members in the analysis.

### Table C23.1

<table>
<thead>
<tr>
<th>Member type</th>
<th>$\tau_{g}$</th>
<th>$n_{eff}[a]$</th>
<th>$P_r \leq P_{ns}$</th>
<th>$P_r &gt; P_{ns}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rolled or built-up I-shaped sections buckling about the minor axis, and other sections not specified in this table</td>
<td>0.50</td>
<td>$n - 3$</td>
<td>$n + 5$</td>
<td></td>
</tr>
<tr>
<td>Rolled or built-up I-shaped sections buckling about the major axis, and welded box sections</td>
<td>0.60</td>
<td>$n - 2$</td>
<td>$n$</td>
<td></td>
</tr>
<tr>
<td>Rectangular HSS</td>
<td>0.60</td>
<td>$n$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Round HSS</td>
<td>0.60</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

[a] $P_r > P_{ns}$ can arise when Appendix 2 is utilized.

(c) Where components comprised of materials other than structural 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.

*Specification for Structural Stainless Steel Buildings, xxx*

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*AMERICAN INSTITUTE OF STEEL CONSTRUCTION*
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, $P_n/\Omega t$, 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, using the appropriate reduction coefficient.

The design tensile strength, $\phi t P_n$, and the allowable tensile strength, $P_n/\Omega t$, 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 using the appropriate reduction coefficient. 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

\[ P_n = F_y A_g \]

(D2-1)

For austenitic and duplex stainless steel tension members
\[ \phi_t = 0.90 \text{ (LRFD)} \quad \Omega_t = 1.67 \text{ (ASD)} \]

For precipitation hardening stainless steel tension rods
\[ \phi_t = 0.80 \text{ (LRFD)} \quad \Omega_t = 1.88 \text{ (ASD)} \]

(b) For tensile rupture in the net section
\[ P_n = F_u A_e \quad \text{(D2-2)} \]

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

**User Note:** Due to the potential for larger ductility and strain hardening in some stainless steels, the fracture in the net section limit state may be associated with large deformation. Designs requiring smaller deformation may limit the stress in the net section to \( f_{\text{max}} \) smaller than \( F_u \), determined based on the maximum member elongation, \( \Delta \), as:

\[ f_{\text{max}} = F_y + E_{sh} \left( \frac{\Delta}{L} \frac{F_y}{E} \right) \]

where
\[ E = \text{modulus of elasticity of stainless steel} \]
\[ = 28,000 \text{ ksi (193 000 MPa) for austenitic, 29,000 ksi (200 000 MPa) for duplex, and 28,500 ksi (197 000) for precipitation hardening stainless steel} \]
\[ E_{sh} = \text{strain hardening modulus determined according to Equation A-2-2, ksi (MPa)} \]
\[ L = \text{length of the tensile member, in. (mm)} \]

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

**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, \( A_e \), of austenitic and duplex stainless steel tension members shall be determined in accordance with the provisions of Section B4.12.

The effective net area of tension members shall be determined as
\[ A_c = A_e U \]  

(D3-1)

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

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

**D4. BUILT-UP MEMBERS**

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

**User Note:** The longitudinal spacing of connectors between components should preferably limit the slenderness ratio in any component between the connectors to 300.
## TABLE D3.1
Shear Lag Factors for Connections to Tension Members

<table>
<thead>
<tr>
<th>Case</th>
<th>Description of Element</th>
<th>Shear Lag Factor, $U$</th>
<th>Examples</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>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).</td>
<td>$U = 1.0$</td>
<td>—</td>
</tr>
<tr>
<td>2</td>
<td>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.)</td>
<td>$U = 1 - \frac{\bar{x}}{l}$</td>
<td><img src="image1" alt="Diagram" /></td>
</tr>
<tr>
<td>3</td>
<td>All tension members where the tension load is transmitted only by transverse welds to some but not all of the cross-sectional elements.</td>
<td>$U = 1.0$ and $A_n = \text{area of the directly connected elements}$</td>
<td>—</td>
</tr>
<tr>
<td>4[a]</td>
<td>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 $\bar{x}$.</td>
<td>$U = \frac{3l^2}{3l^2 + w^2} \left(1 - \frac{\bar{x}}{l}\right)$</td>
<td><img src="image2" alt="Diagram" /></td>
</tr>
<tr>
<td>5</td>
<td>Round and rectangular HSS with single concentric gusset through slots in the HSS.</td>
<td>$\bar{x} = R \sin \theta - \frac{1}{2} l_p \theta$ in rad $U = \left[1 + \frac{\bar{x}}{l}\right]^{-10}$</td>
<td><img src="image3" alt="Diagram" /></td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\bar{x} = \frac{2h^2 + 4H - 2l^2}{2H + 4b - 4t}$ $U = 1 - \frac{\bar{x}}{l}$</td>
<td>—</td>
</tr>
<tr>
<td>6</td>
<td>Rectangular HSS with two side gusset plates.</td>
<td>$U = \frac{BU_B + HU_H}{H + B}$ $U_B = \frac{3l^2}{3l^2 + B^2}$ $U_H = \frac{3l^2}{3l^2 + H^2}$</td>
<td><img src="image4" alt="Diagram" /></td>
</tr>
<tr>
<td>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.)</td>
<td>with flange connected with three or more fasteners per line in the direction of loading $b_f \geq \frac{2}{3}d$, $U = 0.90$</td>
<td>$b_f &lt; \frac{2}{3}d$, $U = 0.85$</td>
<td></td>
</tr>
<tr>
<td>---</td>
<td>---</td>
<td>---</td>
<td></td>
</tr>
<tr>
<td></td>
<td>with web connected with four or more fasteners per line in the direction of loading</td>
<td>$U = 0.70$</td>
<td></td>
</tr>
<tr>
<td>Single and double angles (If $U$ is calculated per Case 2, the larger value is permitted to be used.)</td>
<td>with four or more fasteners per line in the direction of loading</td>
<td>$U = 0.80$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>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)</td>
<td>$U = 0.60$</td>
<td></td>
</tr>
</tbody>
</table>

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

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

### D5. PIN-CONNECTED MEMBERS

#### 1. Tensile Strength

The design tensile strength, $\phi_{P_{\alpha}}$, and the allowable tensile strength, $P_a \Omega_t$, 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, use Section D2(a).

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

$$A_e = 2bh_c \quad (D5-1)$$

(c) For shear rupture on the effective area

$$P_s = 0.6F_a A_{sf} \quad (D5-2)$$

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

where

- $A_{sf} =$ area on the shear failure path, in.$^2$ (mm$^2$)
- $a =$ 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)
- $b_c =$ 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, use Section J7.

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 1.33\(b_e\).

(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:

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. 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.

E1. GENERAL PROVISIONS

The design compressive strength, \( \phi \), \( P_n \), and the allowable compressive strength, \( P_n/\Omega \), 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.

\[ \phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)} \]

User Note: For columns with short unbraced lengths, when applicable, 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

<table>
<thead>
<tr>
<th>Cross Section</th>
<th>Without Slender Elements</th>
<th>With Slender Elements</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Sections in Chapter E</td>
<td>Limit States</td>
</tr>
<tr>
<td>E3</td>
<td>E4</td>
<td>FB</td>
</tr>
<tr>
<td></td>
<td></td>
<td>TB</td>
</tr>
<tr>
<td>E3</td>
<td>E4</td>
<td>FB</td>
</tr>
<tr>
<td></td>
<td></td>
<td>FTB</td>
</tr>
<tr>
<td>E3</td>
<td>FB</td>
<td>E7</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>E6</td>
<td>E3</td>
<td>E4</td>
</tr>
<tr>
<td></td>
<td>E6</td>
<td>FTB</td>
</tr>
<tr>
<td>E5</td>
<td></td>
<td>E7</td>
</tr>
</tbody>
</table>

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

### E2. EFFECTIVE LENGTH

The effective length, \( L_e \), for calculation of member slenderness, \( L_e / r \), shall be determined in accordance with Chapter C,

where
**User Note:** For members designed on the basis of compression, the effective slenderness ratio, \( \frac{L_c}{r} \), preferably should not exceed 200.

**User Note:** The effective length, \( L_c \), can be determined through methods other than 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:

\[
P_n = F_{cr} A_g
\]

(E3-1)

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

(a) When \( \frac{L_c}{r} \leq \frac{\beta_0}{F_y} \left( \frac{E}{F_y} \right) \) (or \( \frac{F_y}{F_e} \leq \left( \frac{\beta_0}{\pi} \right)^2 \))

\[
F_{cr} = F_y
\]

(E3-2)

(b) When \( \beta_0 \frac{E}{F_y} < \frac{L_c}{r} \leq 5.62 \frac{E}{F_y} \) (or \( \left( \frac{\beta_0}{\pi} \right)^2 < \frac{F_y}{F_e} \leq 3.20 \))

\[
F_{cr} = 1.2 \left( \frac{F_y}{F_e} \right)^{\frac{1}{n}}
\]

(E3-3)

(c) When \( \frac{L_c}{r} > 5.62 \frac{E}{F_y} \) (or \( \frac{F_y}{F_e} > 3.20 \))

\[
F_{cr} = \beta_2 F_e
\]

(E3-4)

where

- \( A_g \) = gross area of member, in.\(^2\) (mm\(^2\))
- \( E \) = modulus of elasticity of stainless steel
- \( E \) = 28,000 ksi (193 000 MPa) for austenitic, and 29,000 ksi (200 000 MPa) for duplex stainless steel
**Table E3.1**

<table>
<thead>
<tr>
<th>Member type</th>
<th>Alloy family</th>
<th>Curve</th>
<th>α</th>
<th>β₀</th>
<th>β₁</th>
<th>β₂</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rolled or built-up I-shaped sections buckling about the minor axis, and other sections not specified in this table</td>
<td>Austenitic and Duplex</td>
<td>A</td>
<td>0.565</td>
<td>0.7566</td>
<td>0.40923</td>
<td>0.69750</td>
</tr>
<tr>
<td>Rolled or built-up I-shaped sections buckling about the major axis, welded box sections, and round HSS</td>
<td>Austenitic and Duplex</td>
<td>B</td>
<td>0.58</td>
<td>0.891</td>
<td>0.455</td>
<td>0.820</td>
</tr>
<tr>
<td>Rectangular HSS</td>
<td>Austenitic and Duplex</td>
<td>C</td>
<td>0.69</td>
<td>1.195</td>
<td>0.501</td>
<td>0.820</td>
</tr>
</tbody>
</table>

**User Note:** The two inequalities for calculating the limits of applicability of Sections E3(a) and E3(b), one based on \( \sqrt{\frac{cLr}{yF}} \) and one based on \( \frac{F_y}{F_c} \), 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:

\[
P_n = F_{cr}A_g
\]

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, \( F_c \), determined as follows:

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

\[
F_c = \left( \frac{F_{cy} + F_{cz}}{2H} \right) \left[ 1 - \sqrt{1 - \frac{4F_{cy}F_{cz}H}{(F_{cy} + F_{cz})^2}} \right] 
\]  
(E4-3)

**User Note:** For singly symmetric members with the \( x \)-axis as the axis of symmetry, such as channels, Equation E4-3 is applicable with \( F_{cy} \) replaced by \( F_{cx} \).

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

\[
(C_w - F_{cy})(F_{cx} - F_{cz})(C_c) F_{eq}^2 (C_c - F_{cz}) (\frac{y_c}{t_y})^2 F^{2} (F_c - F_{cy}) (\frac{y_c}{t_y})^2 = 0 \]  
(E4-4)

where

\[
C_w = \text{warping constant, in.}^4 (\text{mm}^4) 
\]

\[
F_{eq} = \left( \frac{\pi^2 Ec}{L_{c}\epsilon} \right) \left( \frac{t_y}{t_x} \right) 
\]  
(E4-5)

\[
F_{cy} = \left( \frac{\pi^2 Ec}{L_{c}\epsilon} \right) \left( \frac{t_y}{t_x} \right) 
\]  
(E4-6)

\[
F_{cz} = \left( \frac{\pi^2 Ec}{L_{c}\epsilon} + GJ \right) \left( \frac{1}{A_g F_0} \right) 
\]  
(E4-7)

\[
G = \text{shear modulus of elasticity of stainless steel, ksi (MPa)} 
\]

\[
10,800 \text{ ksi (74 500 MPa) for austenitic, and 11,200 ksi (77 200 MPa) for duplex stainless steel} 
\]

\[
H = \text{flexural constant} 
\]  
(E4-8)

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

\[
J = \text{torsional constant, in.}^4 (\text{mm}^4) 
\]

\[
K_x = \text{effective length factor for flexural buckling about x-axis} 
\]

\[
K_y = \text{effective length factor for flexural buckling about y-axis} 
\]

\[
K_z = \text{effective length factor for torsional buckling about the longitudinal axis} 
\]

\[
L_{et} = \text{effective length of member for buckling about x-axis, in.} (\text{mm}) 
\]

\[
K_L 
\]

---

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Public Review draft dated October 14, 2020

**AMERICAN INSTITUTE OF STEEL CONSTRUCTION**
Effective length of member for buckling about y-axis, in. (mm)  
\[ L_{ey} = K_y L_y \]

Effective length of member for buckling about longitudinal axis, in. (mm)  
\[ L_{el} = K_z L_z \]

Laterally unbraced length of the member for the y- and longitudinal axes, in. (mm)  
\[ L_{u}, L_y, L_z = \text{laterally unbraced length of the member for the y- and longitudinal axes} \]

Polar radius of gyration about the shear center, in. (mm)  
\[ r_o^2 = x_o^2 + y_o^2 + I_x + I_y \]

Radius of gyration about x-axis, in. (mm)  
\[ r_x = \text{radius of gyration about } x\text{-axis} \]

Radius of gyration about y-axis, in. (mm)  
\[ r_y = \text{radius of gyration about } y\text{-axis} \]

Coordinates of the shear center with respect to the centroid, in. (mm)  
\[ x_o, y_o = \text{coordinates of the shear center with respect to the centroid} \]

User Note: For doubly symmetric I-shaped sections, \( C_o \) may be taken as  
\[ I_{ho}^2 / 4 \], where \( h_o \) is the distance between flange centroids, in lieu of a more precise analysis. For tees and double angles, the term with \( C_o \) may be omitted when computing \( F_{eq} \).

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. \[ L_e / r_e \] 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 \[ \frac{L}{r_o} \leq 80 \]
\[
\frac{L_c}{r} = 72 + 0.75 \frac{L}{r_a} \quad \text{(E5-1)}
\]

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

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

(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 \( \frac{L}{r_a} \leq 75 \)

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

(2) When \( \frac{L}{r_a} > 75 \)

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

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)
- \( r_a \) = radius of gyration about the geometric axis parallel to the connected leg, 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 \( (L_c/r)_m \), determined as follows:

(a) For intermediate connectors that are bolted snug-tight
(b) For intermediate connectors that are welded or are connected by means of pretensioned bolts with Class A or B faying surfaces as specified in Table J3.4.

(1) When \( \frac{a}{r_i} \leq 40 \)

\[
\left( \frac{L_c}{r_m} \right) = \left( \frac{L_c}{r_o} \right) \]  
(E6-2a)

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

\[
\left( \frac{L_c}{r_m} \right) = \left( \frac{L_c}{r_o} \right)^2 + \left( \frac{K_i a}{r_i} \right)^2 \]  
(E6-2b)

where

\[
\left( \frac{L_c}{r_m} \right) \quad \text{modified slenderness ratio of built-up member}
\]

\[
\left( \frac{L_c}{r_o} \right) \quad \text{slenderness ratio of built-up member acting as a unit in the buckling direction being addressed}
\]

\( L_c \) = 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, \( a/r_s \), 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.

$$P_n = F_{cr} A_e$$

where

- $A_e = \text{summation of the effective areas of the cross section based on reduced effective widths, } b_e, d_e \text{ or } h_e, \text{ in.}^2 (\text{mm}^2)$
- $F_{cr} = \text{critical stress determined in accordance with Section E3 or E4, ksi (MPa)}$

**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 $(b - b_e)\ell$.

Channels in compression with slender flanges and pinned restraints at the ends shall be designed according to Section H1.1 for combined axial force and flexure about the minor axis resulting from the shift of the effective centroid.

**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. (25.4 mm) are given in Specification for the Design of Cold-Formed Stainless Steel Structural Members (ASCE/SEI 8).

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:

(1a) When $\lambda \leq \lambda_y$,

$$b_e = \frac{F_y}{F_{cr}}$$

(2b) When $\lambda > \lambda_y$,

$$b_e = 0.772b \left[1 - 0.10 \left(\frac{F_{el}}{F_{cr}}\right) \right]$$

(1) For stiffened elements:

(2) For unstiffened elements:
where

\[ E = \text{modulus of elasticity of stainless steel} \]

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

\[ F_{el} = \text{elastic local buckling stress of the full cross section, or conserva-} \]

tively determined according to Equation E7-4 for each individual element of the cross-section, ksi (MPa)

\[
F_{el} = \frac{-\pi^2 E}{12(1-\nu^2)} \left( \frac{1}{\lambda} \right)^2 \left( \frac{2}{\lambda_r} \right)^2 F_y
\]

\[ F_y = \text{specified minimum yield stress, which for cold-formed square or rectangular HSS, may be replaced with } F_{y,avg} \text{ determined in accordance with Section B4.34, ksi (MPa)} \]

\[ b = \text{width of the element (for tees stems this is } d; \text{ for webs this is } h), \text{ in. (mm)} \]

\[ k = \text{plate buckling coefficient} \]

- 0.425 for unstiffened elements in compression
- 4.00 for stiffened elements in compression

\[ \lambda = \text{width-to-thickness ratio for the element as defined in Section B4.1} \]

\[ \lambda_r = \text{limiting width-to-thickness ratio as defined in Table B4.1a} \]

\[ \nu = \text{Poisson’s ratio} = 0.3 \]

**User Note:** The Commentary 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, the effective width, \( b_e \), is permitted to be determined as follows:

\[ \phi = 0.85 \text{ (LRFD)} \]

\[ \Omega = 1.76 \text{ (ASD)} \]

(1) When \( \lambda \leq \lambda_r \)

\[
b_e = b \left( 1 - 0.19 \frac{F_{el}}{F_{cr}} \right) \left( \frac{F_{el}}{F_{cr}} \right)
\]

(2) When \( \lambda > \lambda_r \)

\[
b_e = b \left( 1 - 0.22 \frac{F_{el}}{F_{cr}} \right) \left( \frac{F_{el}}{F_{cr}} \right)
\]

If Equations E7-5 and E7-6 are used to calculate \( b_e \), then the use of \( F_{y,avg} \) for cold-formed square and rectangular HSS when calculating \( b_e \) or \( F_{cr} \) is not permitted.

**User Note:** For cold-formed HSS with slender elements, if the increase in yield strength resulting from cold-work of forming (\( F_{y,avg} \)) is not considered, the effective width may be determined using an alternative expression given in the Commentary.
2. **Round HSS**

The effective area, \( A_e \), is determined as follows:

(a) When \( \lambda \leq \lambda_r \frac{F_y}{F_{cr}} \)

\[ A_e = A_g \]  
(E7-76)

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

\[ A_e = \frac{3 \lambda_r + 2 \lambda}{5 \lambda} A_g + \frac{0.6 \lambda_r}{\lambda} \left( \frac{F_y}{F_{cr}} + \frac{2}{5} \right) A_g \]  
(E7-87)

where \( D \) = outside diameter of round HSS, in. (mm)  
\( F_y \) = specified minimum yield stress, which for cold-formed round HSS, may be replaced with \( F_{y,avg} \) determined in accordance with Section B4.32, ksi (MPa)  
\( t \) = design wall thickness of wall, as defined in Section B4.2, in. (mm)  
\( \lambda = \frac{D}{t} \)  
\( \lambda_r \) = 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 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. Rectangular Bars 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

<table>
<thead>
<tr>
<th>Section in Chapter F</th>
<th>Cross Section</th>
<th>Flange Slenderness</th>
<th>Web Slenderness</th>
<th>Limit States</th>
</tr>
</thead>
<tbody>
<tr>
<td>F2</td>
<td>C</td>
<td>C</td>
<td></td>
<td>Y, LTB</td>
</tr>
<tr>
<td>F3</td>
<td>NC, S</td>
<td>C</td>
<td></td>
<td>LTB, FLB</td>
</tr>
<tr>
<td>F4</td>
<td>C, NC, S</td>
<td>C, NC</td>
<td></td>
<td>CFY, LTB, FLB</td>
</tr>
<tr>
<td>F5</td>
<td>C, NC, S</td>
<td>S</td>
<td></td>
<td>CFY, LTB, FLB</td>
</tr>
<tr>
<td>F6</td>
<td>C, NC, S</td>
<td>N/A</td>
<td></td>
<td>Y, FLB</td>
</tr>
<tr>
<td>F7</td>
<td>C, NC, S</td>
<td>C, NC, S</td>
<td></td>
<td>Y, FLB, WLB</td>
</tr>
<tr>
<td>F8</td>
<td>N/A</td>
<td>N/A</td>
<td></td>
<td>Y, LB</td>
</tr>
<tr>
<td>F9</td>
<td>N/A</td>
<td>N/A</td>
<td></td>
<td>Y</td>
</tr>
<tr>
<td>F10</td>
<td>Other shapes, other than single angles</td>
<td>N/A</td>
<td>N/A</td>
<td>All limit states</td>
</tr>
</tbody>
</table>

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, $M_a/\Omega_b$, shall be determined as follows:

(a) For all provisions in this chapter

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

and the nominal flexural strength, $M_n$, shall be determined according to Sections F2 through 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. (25.4 mm) are given in the
The lateral-torsional buckling modification factor, $C_b$, for nonuniform moment diagrams when both ends of the segment are braced is determined as follows:

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

where

- $M_{\text{max}}$ = absolute value of maximum moment in the unbraced segment, kip-in. (N-mm)
- $M_A$ = absolute value of moment at quarter point of the unbraced segment, kip-in. (N-mm)
- $M_B$ = absolute value of moment at centerline of the unbraced segment, kip-in. (N-mm)
- $M_C$ = absolute value of moment at three-quarter point of the unbraced segment, kip-in. (N-mm)

**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 to the Specification for Structural Steel Buildings, hereafter referred as the 2016 AISC Specification. The Commentary to 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, $C_b = 1.0$.

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.
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**

   \[ M_n = M_p = F_y Z_x \]  \( \text{(F2-1)} \)

   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 \( L_p \leq L_r \), the limit state of lateral-torsional buckling does not apply.

   (b) When \( L_p < L_p \leq L_y \)

   \[ M_n = C_b \left[ M_p - (M_p - F_y S_x) \left( \frac{L_p - L_y}{L_y - L_p} \right) \right] \leq M_p \]  \( \text{(F2-2)} \)

   (c) When \( L_y < L_y \leq L_y \)

   \[ M_n = C_b \left[ M_y - (M_y - 0.30F_y S_y) \left( \frac{L_y - L_p}{L_p - L_y} \right) \right] \leq M_y \]  \( \text{(F2-3)} \)

   (d) When \( L_p > L_p \)

   \[ M_n = \beta_{LT} F_c, S_x \leq M_y \]  \( \text{(F2-4)} \)

where

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

\( F_c, = \) critical stress, ksi (MPa)

\( \beta_{LT} = \) elastic lateral-torsional buckling reduction coefficients determined from Table F2.1

\( E = \) modulus of elasticity of stainless steel

\( J_c = 28,000 \text{ ksi (193 000 MPa) for austenitic, and 29,000 ksi (200 000 MPa) for duplex stainless steel} \)

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

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

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

\( \alpha_{LT} = 0.60 - 0.40 \frac{L_p - L_y}{L_y - L_p} \)  \( \text{(F2-5)} \)
User Note: The square root term in Equation F2-56 may be conservatively taken equal to 1.0.

### Table F2.1
Lateral-Torsional Buckling Coefficients

<table>
<thead>
<tr>
<th>Alloy Family</th>
<th>$\beta_{LT}$</th>
<th>$\beta_{p,LT}$</th>
<th>$\beta_{y,LT}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Austenitic</td>
<td>0.82</td>
<td>0.90</td>
<td>0.40</td>
</tr>
<tr>
<td>Duplex</td>
<td>0.86</td>
<td>1.10</td>
<td>0.50</td>
</tr>
</tbody>
</table>

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

$$L_p = \beta_{p,LT} r_o \sqrt{\frac{E}{F_y}}$$  \hfill (F2-7)

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

$$L_y = 1.95 \beta_{y,LT} r_o \sqrt{\frac{J_c}{S_x h_o} + \left( \frac{J_c}{S_y h_o} \right)^2 + 6.76 \left( \frac{F_y}{\beta_{LT} E} \right)^2}$$  \hfill (F2-8)

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

$$L_r = 1.95 r_o \sqrt{\frac{J_c}{0.30 F_y} + \left( \frac{J_c}{S_y h_o} \right)^2 + 6.76 \left( \frac{0.30 F_y}{\beta_{LT} E} \right)^2}$$  \hfill (F2-9)

where

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

$$r_o^2 = \frac{I_y C_w}{S_x}$$  \hfill (F2-10)

and the coefficient $c$ is determined as follows:

1. For doubly symmetric I-shapes

$$c = 1$$  \hfill (F2-11a)

2. For channels

$$c = \frac{h_o}{2} \sqrt{\frac{I_y}{C_w}}$$  \hfill (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, \( C_w = \frac{I_y h_w^2}{4} \)
and thus, Equation F2-10 becomes
\[
r_s^2 = \frac{I_y h_w}{2S_y}
\]
\( r_s \) may be approximated accurately and conservatively as the radius of gyration of the compression flange plus one-sixth of the web:
\[
r_s = \frac{b_f}{\sqrt{1 + \frac{b_{ef} h_w}{6 b_f t_f}}}
\]

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

\[
M_n = M_p - (M_p - F_y S_{ef}) \left( \frac{\lambda - \lambda_{pf}}{\lambda_{pf}} \right)
\]

(b) For sections with slender flanges

\[
M_n = F_y S_{ef}, \quad M_n = F_y S_{ef}
\]

where

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

\[
b_{ef} = b_f \left( 1 - 0.19 \sqrt{\frac{F_{el}}{F_y}} \right) \sqrt{\frac{F_{el}}{F_y}} \leq b_f, \quad b_c = 0.7726 \left( 1 - 0.10 \sqrt{\frac{F_{el}}{F_y}} \right) \sqrt{\frac{F_{el}}{F_y}} \leq b \quad (F3-3)
\]
$F_{ed} = \text{elastic local buckling stress of the full cross section, or conservatively determined for the compression flange according to Equation F3-4 for the compression flange, ksi (MPa)}$

$$F_{ed} = \frac{k \cdot \pi^2 E}{12(1-\nu^2)} \left(1 - \frac{\lambda_{eff}}{\lambda}ight) \left(1.53 \frac{\lambda_{ef}}{\lambda}ight)^2 F_y$$

(F3-4)

- $b_f = \text{width of the flange, in. (mm)}$
- $b = \text{width of element; for flanges of I-shaped members, half the full flange width, } b_f$; for flanges of channels, the full nominal dimension of the flange, in. (mm)$
- $k = \text{plate buckling coefficient}$
- $\lambda = \frac{2 \nu}{b}$

User Note: The Commentary 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.

Alternatively, it is permitted to calculate the effective width of the compression flange, $b_e$, using Equation F3-5:

$$b_e = b \left(1 - 0.22 \frac{F_{ed}}{F_y} \right) \left(1.53 \frac{\lambda_{ef}}{\lambda}ight) \leq b \quad \text{(F3-5)}$$

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

$$M_n = R_p M_o \quad \text{(F4-1)}$$

where

$$M_o = F_o S_o = \text{yield moment about the axis of bending, kip-in. (N-mm)}$$

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2. Lateral-Torsional Buckling

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

(b) When \( L_p < L_b \leq L_y \)

\[
M_u = C_b \left[ R_p M_y - \left( R_p M_y - F_y S_y \right) \left( \frac{L_b - L_p}{L_y - L_p} \right) \right] \leq R_p M_y \quad \text{(F4-2)}
\]

(c) When \( L_y < L_b \leq L_r \)

\[
M_u = C_b \left[ M_y - \left( M_y - 0.30 F_y S_y \right) \left( \frac{L_b - L_y}{L_r - L_y} \right)^{\alpha_{LT}} \right] \leq M_y \quad \text{(F4-3)}
\]

(d) When \( L_b > L_r \)

\[
M_u = \beta_{LT} F_y S_y \leq R_p M_y \quad \text{(F4-4)}
\]

where

\( \alpha_{LT} \) = lateral-torsional buckling coefficient determined by Equation F2-65

\( \beta_{LT} \) = elastic lateral-torsional buckling reduction coefficient determined from Table F2.1

\( F_y \) = critical stress, determined by Equation F2-56

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

\( L_y \) = 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

\( R_p \), the web plastification factor, is determined as follows:

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

\[
R_p = \left[ \frac{M_p}{M_y} \left( \frac{M_p}{M_y} - 1 \right) \left( \frac{\lambda}{\lambda_{pw}} - 1 \right) \right] \leq \frac{M_p}{M_y} \quad \text{(F4-5a)}
\]

2. When \( I_{yf}/I_y \leq 0.23 \)

\[
R_p = 1.0 \quad \text{(F4-5b)}
\]

where

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

\( M_p = F_y Z_y \)

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

\( \lambda = \frac{h}{t_w} \)

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

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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

\[
M_u = R_p M_y - \left( R_p M_y - F_s S_y \right) \left( \frac{\lambda - \lambda_{pf}}{\lambda_{pf} - \lambda_{pf}} \right) \quad \text{(F4-640)}
\]

(c) For sections with slender flanges

\[
M_u = F_s S_y M_y = F_s S_{wy} \quad \text{(F4-744)}
\]

where

- \( S_{wy} \) = 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, \( b_{ye} \), 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, \( \text{in.}^3 \text{ (mm}^3 \text{)} \)

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

- \( \lambda = \frac{b_y}{2t_y} \)

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

- \( \lambda_{nf} \) = 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, \( M_u \), 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

\[
M_u = R_p F_s S_y \quad \text{(F5-1)}
\]

2. Lateral-Torsional Buckling

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

(b) When \( L_y < L_b \leq L_r \)
\[ M_u = C_s R_{pg} \left[ M_y - \left( M_y - 0.30 F_y S_y \right) \left( \frac{L_y - L_y}{L_y - L_y} \right)^{\alpha_{lt}} \right] \leq M_y \] (F5-2)

(c) When \( L_y > L_r \)
\[ M_u = \beta_{lt} R_{pg} F_{cr} S_x \leq M_y \] (F5-3)

where
- \( \alpha_{lt} \) = lateral-torsional buckling coefficient determined by Equation F2-64
- \( \beta_{lt} \) = elastic lateral-torsional buckling reduction coefficient determined from Table F2.1
- \( F_{cr} \) = critical stress, determined by Equation F2-56
- \( L_y \) = 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

\( R_{pg} \), the bending strength reduction factor, is:
\[ R_{pg} = 1 - \frac{a_w}{6.75 + 2.12 a_w} \left( \frac{\lambda}{\lambda_{cr}} - 1.0 \right) \leq 1.0 \] (F5-4)

where
- \( a_w = \frac{h t_w}{b_y t_f} \leq 10.0 \) (F5-5)
- \( b_y \) = 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)
- \( \lambda = \frac{h}{t_w} \)
- \( \lambda_{cr} = \lambda_{cr} \), 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
\[ M_u = R_{pg} F_{cr} S_{pe} \]
\[ M_u = R_{pg} F_{cr} S_{pe} \] (F5-6)

where
- \( S_{pe} \) = 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\(^3\))
- \( R_{pg} \) = bending strength reduction factor, determined by Equation F5-4
**F6. I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MINOR AXIS**

This section applies to I-shaped members and channels bent about their minor axis.

The nominal flexural strength, $M_n$, shall be the lower value obtained according to the limit states of yielding (plastic moment) and flange local buckling.

1. **Yielding**

   $$M_n = M_p = F_yZ_y$$  \hspace{1cm} (F6-1)

   where

   $S_{ey} = $ elastic section modulus taken about the y-axis, in.³ (mm³)
   
   $Z_y = $ plastic section modulus taken about the y-axis, in.³ (mm³)

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

   $$M_n = M_p - (M_p - F_y S_{ey}) \left( \frac{\lambda - \lambda_{pf}}{\lambda_{ef} - \lambda_{pf}} \right)$$  \hspace{1cm} (F6-2)

   (c) For sections with slender flanges

   $$M_n = F_y S_{ye}$$  \hspace{1cm} (F6-3)

   where

   $S_{ye} = $ elastic section modulus referred to the extreme compressive fibers, in.³ (mm³)

   $S_{ye}$ = effective section modulus referred to the extreme compressive fibers taken about the y-axis with respect to the neutral axis of the effective cross section determined with the effective width, $b_e$, of the compression flanges 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 assuming they are subject to uniform compression.

   $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)

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

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

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\( \lambda_{ef} = \lambda_r \), 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

\[
M_u = M_p - \left( M_p - F_y S_y \right) \left( \frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}} \right) \quad (F6-4)
\]

(c) For channel with slender webs when \( \lambda_{pw} < \lambda \leq 1.5 \lambda_{pw} \)

\[
M_u = F_y S_y \quad (F6-5)
\]

where

- \( S_y \) = elastic section modulus referred to the extreme tensile fibers, in.\(^3\)
- \( h \) = 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)
- \( \lambda = \frac{h}{t_w} \)
- \( \lambda_{pw} = \lambda_p \), 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 with \( h/b \leq 3 \), and doubly symmetric box sections bent about either axis, having compact, noncompact, or slender webs or flanges, as defined in Section B4.1 for flexure.

The nominal flexural strength, \( M_{nf} \), 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 with \( h/b \leq 3 \) will not be susceptible to lateral-torsional buckling. For longer lengths, beam deflection is likely to be critical.

**User Note:** For cold-formed HSS with slender elements, if the increase in yield strength resulting from cold-work of forming \( (F_{y,\text{avg}}) \) is not considered, the effective width of the webs and flanges may be determined using an alternative expression given in the Commentary.

#### 1. Yielding

\[
M_u = M_p = F_y Z \quad (F7-1)
\]

where

\[ F_n \]
2. Flange Local Buckling

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

(b) For sections with noncompact flanges

\[ M_u = M_p - (M_p - F_y S) \left( \frac{\lambda - \lambda_{pf}}{\lambda_{pf} - \lambda_{p}} \right) \leq M_p \]  

(F7-2)

(c) For sections with slender flanges

\[ M_u = F_y S_e \]  

(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 taken as 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

\[ h_y = 0.772 \left( 1 - 0.10 \left( \frac{F_{el}}{F_y} \right)^2 \right) \]  

(F7-4)

\[ F_{el} = \text{elastic local buckling stress of the full cross section, or conservatively determined according to Equation F7-5 for the compression flange, ksi (MPa)} \]

\[ b = \text{width of compression flange as defined in Section B4.1b, in. (mm)} \]

\[ k = \text{plate buckling coefficient} \]

\( k = 4.00 \) for the compression flange of square and rectangular HSS, and box sections

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

\[ \lambda = \frac{b}{t_f} \]

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

\( \lambda_{pf} = \lambda_{p} \), the limiting width-to-thickness ratio for a noncompact flange, defined in Table B4.1b

\[ \nu = \text{Poisson's ratio} = 0.3 \]
User Note: The Commentary gives analytical expressions for determining the elastic local buckling stress for the full cross section. Alternatively, the elastic buckling stress of the full cross section may be determined using numerical methods.

User Note: For cold-formed HSS with slender elements, if the increase in yield strength resulting from cold-work of forming \((F_{y,avg})\) is not considered, the effective width may be determined using an alternative expression given in the Commentary.

3. Web Local Buckling

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

(b) For sections with noncompact webs

\[
M_n = M_p - (M_p - F_y S) \left( \frac{\lambda - \lambda_{pw}}{\lambda_{pw} - \lambda_{p}} \right) \leq M_p 
\] (F7-6)

(c) For sections with slender webs

(1) Compression flange yielding

\[
M_n = R_{ps} F_y S
\] (F7-7)

(2) Compression flange local buckling

\[
M_n = R_{ps} F_y S_e
\] (F7-8)

where

\[
R_{ps} = \text{bending strength reduction factor, determined by Equation F5-4 with } \frac{a_w}{a_w} = \frac{1}{h(t_w)}
\]

\[
S_e = \text{effective section modulus, as defined in Section F7.2 with respect to the neutral axis of the effective cross section determined with the effective width, } b_e, \text{ of the compression flange given by Equation F7.4}
\]

\[
h = \text{depth of web, as defined in Section B4.1b, in. (mm)}
\]

\[
t_w = \text{design thickness of the web, as defined in Section B4.2, in. (mm)}
\]

\[
\lambda = \frac{h}{t_w}
\]

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

\[
\lambda_{re} = \text{the limiting width-to-thickness ratio for a noncompact web, defined in Table B4.1b}
\]

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.

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1. Yielding

\[ M_n = M_p = F_y Z \]  

(F8-1)

2. Local Buckling

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

(b) For noncompact sections

\[ M_n = \left( \frac{0.068 \lambda_p}{\lambda} + 1 \right) F_y S \]  

(F8-2)

where

- \( \lambda = \frac{D}{t} \)
- \( \lambda_p \) = limiting width-to-thickness ratio as defined in Table B4.1b

F9. RECTANGULAR BARS AND ROUNDS

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

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

1. Yielding

For rectangular bars

\[ M_n = M_p = F_y Z \]  

(F9-1)

For rounds

\[ M_n = M_p = F_y Z \]  

(F9-2)

2. Lateral-Torsional Buckling

(a) For rectangular bars with \( \frac{L_b d}{t^2} \leq \frac{0.31E}{F_y} \) bent about their major axis, rectangular bars bent about their minor axis and rounds, the limit state of lateral-torsional buckling does not apply.

(b) For rectangular bars with \( \frac{0.31E}{F_y} < \frac{L_b d}{t^2} \leq \frac{1.72E}{F_y} \) bent about their major axis

\[ M_n = C_b \left[ 1.61 - 0.355 \left( \frac{L_b d}{t^2} \right) \frac{F_y}{E} \right] M_p \leq M_p \]  

(F9-3)

where

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

(c) For rectangular bars with \( \frac{L_d d}{t^2} > \frac{1.72E}{F_y} \) bent about their major axis

\[ M_n = F_{cr} S_x \leq M_p \quad \text{(F9-4)} \]

where

\[ F_{cr} = \frac{1.72EC_b}{L_d d} \quad \text{(F9-5)} \]

### F10. OTHER SHAPES

This section applies to all unsymmetrical shapes without slender elements except single angles.

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, and local buckling where

\[ M_n = F_y S_{min} \quad \text{(F10-1)} \]

where

\[ S_{min} = \text{minimum elastic section modulus relative to the axis of bending, in.}^2 \text{ (mm}^4) \]

**User Note:** The design provisions within this section can be overly conservative for certain shapes, unbraced lengths and moment diagrams. To improve economy, the provisions of Appendix 1.3 are recommended as an alternative for determining the nominal flexural strength of members of unsymmetrical shape.

#### 1. Yielding

\[ F_y = F_y \quad M_n = F_y S_{min} \quad \text{(F10-12)} \]

where

\[ S_{min} = \text{minimum elastic section modulus relative to the axis of bending, in.}^2 \text{ (mm}^4) \]

#### 2. Lateral-Torsional Buckling

(a) When \( L_d \leq L_y \), the limit state of lateral-torsional buckling does not apply.

(b) When \( L_y < L_d \leq L_y \)

\[ M_n = C_b \left[ F_y S_{min} - \left( F_y S_{min} - 0.30F_y S_{min} \right) \left( \frac{L_d - L_y}{L_y - L_y} \right)^{\alpha_{LT}} \right] \leq F_y S_{min} \quad \text{(F10-2)} \]

(c) When \( L_d > L_y \)

\[ M_n = 0.82F_y S_{min} \leq F_y S_{min} \quad \text{(F10-3)} \]

where

\[ \alpha_{LT} = \text{lateral-torsional buckling coefficient determined by Equation F2-6} \]

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

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

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

\[ F_c \leq F_{cr} \leq F_y \quad (F10-3) \]

where

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

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

### 3. Local Buckling

\[ F_c = F_{cr} \leq F_y \quad (F10-4) \]

where

\[ F_{cr} = \text{local buckling stress for the section as determined by analysis, ksi} \]

#### 1. Strength Reductions for Members with Holes in the Tension Flange

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

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

(a) When \( F_n A_{fg} \geq F_y A_{fg} \), the limit state of tensile rupture does not apply.

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

\[ M_n = \frac{F_n A_{fg}}{A_{fg}} S_x \quad (F11-1) \]

where

\[ A_{fg} = \text{gross area of tension flange, calculated in accordance with the provisions of Section B4.43a, in.}^2 (\text{mm}^2) \]

\[ A_{fn} = \text{net area of tension flange, calculated in accordance with the provisions of Section B4.43b, in.}^2 (\text{mm}^2) \]

\[ F_n = \text{specified minimum tensile strength, ksi (MPa)} \]

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

#### 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

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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 I-shaped members and channels 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:

G2. Doubly Symmetric 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, Doubly Symmetric I-Shaped Members, Channels, 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: Angles, tees, and singly symmetric I-shaped members 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

The design shear strength, \( \phi V_n \), and the allowable shear strength, \( V_n/\Omega_v \), shall be determined as follows:

(a) For all provisions in this chapter except Section G7

\[ \phi_v = 0.90 \text{ (LRFD)} \quad \Omega_v = 1.67 \text{ (ASD)} \]

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

G2. DOUBLY SYMMETRIC 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:

$$ V_n = 0.6 F_y A_w C_{v1} $$  \hspace{1cm} (G2-1)

where

- $F_y$ = specified minimum yield stress, ksi (MPa)
- $A_w$ = area of web, the overall depth times the web design thickness, $d_{tw}$, in.$^2$ (mm$^2$)

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

1. When $\lambda \leq 0.59 \frac{E}{F_y}$
   $$ C_{v1} = 1.2 $$  \hspace{1cm} (G2-2)

2. When $\lambda > 0.59 \frac{E}{F_y}$
   $$ C_{v1} = \frac{1.55 \sqrt{E}}{0.7 \frac{E}{F_y} + \lambda} $$  \hspace{1cm} (G2-3)

where

- $E$ = modulus of elasticity of stainless steel
  - 28,000 ksi (193 000 MPa) for austenitic, and 29,000 ksi (200 000 MPa) for duplex stainless steel
- $h$ = 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)
- $\lambda = \frac{h}{t_w}$

(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
   $$ k_v = 5 + \frac{5}{(a/h)^2} $$  \hspace{1cm} (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 $\lambda \leq 0.65 \sqrt{k_c E / F_y}$

$$V_n = 0.60 C_{v_2} F_y A_w$$  \hspace{1cm} (G2-5)

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

(1) When $2 A_w / (A_c + A_t) \leq 2.5$, $h/b_c \leq 6.0$ and $h/b_t \leq 6.0$

$$V_n = 0.6 F_y A_w \left[ C_{v_2} + \frac{1 - C_{v_2}}{1.15 \sqrt{1 + (a/h)^2}} \right]$$  \hspace{1cm} (G2-6)

(2) Otherwise

$$V_n = 0.6 F_y A_w \left[ C_{v_2} + \frac{1 - C_{v_2}}{1.15 \left( a/h + \sqrt{1 + (a/h)^2} \right)} \right]$$  \hspace{1cm} (G2-7)

where

The web shear buckling coefficient, $C_{v_2}$, is determined as follows:

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

$$C_{v_2} = 1.2$$  \hspace{1cm} (G2-8)

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

$$C_{v_2} = 1.2 - 0.62 \left( \frac{\lambda}{\sqrt{k_c E / F_y}} - 0.33 \right)$$  \hspace{1cm} (G2-9)

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

$$C_{v_2} = \frac{5.02 \sqrt{k_c E / F_y} - \lambda}{1.62 \sqrt{k_c E / F_y} + 3.55 \lambda}$$  \hspace{1cm} (G2-10)

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

$$C_{v_2} = \frac{1.51 k_c E}{\lambda^2 F_y}$$  \hspace{1cm} (G2-11)

$A_c$ = area of compression flange, in.$^2$ (mm$^2$)

$A_t$ = area of tension flange, in.$^2$ (mm$^2$)

$b_c$ = width of compression flange, in. (mm)
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 $\lambda \leq 2.46 \sqrt{\frac{E}{F_{yw}}}$, or where the available shear strength provided in accordance with Section G2.1 for $k_s = 5.34$ 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) $\frac{(h/t)_{st}}{t_{w}} \leq 0.47 \sqrt{\frac{E}{F_{yw}}}$

(e) $I_{st} \geq I_{st1} + (I_{st1} - I_{st2}) \rho_w$

where

- $F_{yw}$ = specified minimum yield stress of the web 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$^4$)
- $I_{st2}$ = minimum moment of inertia of the transverse stiffeners required for development of the web shear buckling resistance, $V_r = V_{c2}$, in.$^4$ (mm$^4$)

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

$\rho_w$ = rib of weld penetration factor

$E$ = Young's modulus of the web material, ksi (MPa)

$F_{yw}$ = specified minimum yield stress of the web material, ksi (MPa)
\[ I_{st} = \frac{2.5}{(a/h)^2} b w^6 + 0.5 b w^6 \] \hspace{1cm} (G2-15)

G3. RECTANGULAR HSS AND BOX SECTIONS SUBJECT TO SHEAR

The nominal shear strength, \( V_n \), of rectangular HSS or box section members shall be determined as:

\[ V_n = 0.6 F_y A_y \] \hspace{1cm} (G3-1)

where

\[ A_y = 2ht, \text{ in.}^2 (\text{mm}^2) \]

\[ C_{v2} = \text{web shear buckling strength coefficient, as defined in Section G2.2,} \]

with \( \lambda = h/t \) and \( k_i = 5 \)

\[ h = \text{width resisting the shear force, taken as the clear distance between the flanges less the inside corner radius on each side for HSS or the clear distance between flanges for box sections, in. (mm). If the corner radius is not known,} \]

\[ t = \text{design wall thickness, as defined in Section B4.2, in. (mm)} \]

G4. ROUND HSS SUBJECT TO SHEAR

The nominal shear strength, \( V_n \), of round HSS, according to the limit states of shear yielding and shear buckling, shall be determined as:

\[ V_n = 0.6 F_y A_y C_{v2} \] \hspace{1cm} (G3-1)
\[ V_s = 0.6 F_y A_g C_v/2 \]  

where

\[ A_g = \text{gross area of member, in.}^2 \text{ (mm}^2) \]
\[ C_v = \text{shall be the larger of } C_{v1} \text{ and } C_{v2} \]
\[ F_y = \text{specified minimum yield stress, ksi (MPa)} \]

The shear strength coefficient, \( C_v \), is determined as follows:

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

\[ C_v = C_{vM} \]  

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

\[ C_v = C_{vL} \]  

where

\[ D = \text{outside diameter, in. (mm)} \]
\[ L_v = \text{distance from the point of maximum shear force to the point of zero shear force, in. (mm)} \]

\[ \lambda = \frac{D}{t} \]

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

(i) When \( \lambda \leq \frac{0.80}{0.32} \)

\[ C_{vM} = 1.0 \]  

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

\[ C_{vM} = 1 - 0.61 \left( 0.47 \sqrt{\frac{L_v/D}{E / F_y}} \right)^{1.25} - 0.23 \]  

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

\[ C_{vM} = \frac{2.67 E}{\sqrt{L_v/D} \lambda^{1.25} F_y} \]  

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

(i) When \( \lambda \leq 0.24 \left( \frac{E / F_y}{2} \right)^{0.67} \)
\[ C_{L2} = 1.0 \] (G4-75)

(ii) When \( 0.24 \left( \frac{E}{F_y} \right)^{0.67} < \lambda \leq 2.23 \left( \frac{E}{F_y} \right)^{0.67} \)

\[ C_{L2} = 1 - 0.61 \left( 0.68 \frac{\lambda^{1.50}}{E/F_y} - 0.23 \right) \] (G4-86)

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

\[ C_{L2} = \frac{1.30E}{\lambda^{1.50}F_y} \] (G4-92)

\[ E \] = modulus of elasticity of stainless steel
\[ D \] = outside diameter, in. (mm)
\[ F_y \] = specified minimum yield stress, ksi (MPa)
\[ k \] = distance from maximum to zero shear force, in. (mm)
\[ t \] = thickness, in. (mm)
\[ \lambda = \frac{D}{t} \]

**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:

\[ V_n = 0.6F_y b_f t_f C_{L2} \] (G5-1)

where

\[ C_{L2} = \text{web shear buckling strength coefficient, as defined in Section G2.2} \]

with \( \lambda = \frac{b_f}{2t_f} \) for I-shaped members, or \( \lambda = \frac{b_f}{t_f} \) 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:

\[ V_n = 0.6F_y A_n C_{L2} \] (G6-1)

where

\[ A_n = \text{area of element or elements resisting the shear force, taken as the sum of the overall depth times the element design thickness, in.}^2 \] (mm²)
\[ C_{v2} = \text{web shear buckling strength coefficient, as defined in Section G2.2,} \]
\[ \lambda = d/t \text{ and } k_v = 5 \text{ for stiffened elements, and } k_v = 1.2 \text{ for unstiffened elements} \]
\[ d = \text{width of element resisting the shear force, in. (mm)} \]
\[ = \text{for built-up welded sections or members, the clear distance between flanges, } h, \text{ in. (mm)} \]
\[ = \text{for built-up bolted members, the distance between fastener lines, } b, \text{ in. (mm)} \]
\[ t = \text{element design thickness, as defined in Section B4.2, in. (mm)} \]

**G76. 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.

**G87. ROUND AND RECTANGULAR HSS, AND BOX SECTIONS SUBJECT TO TORSION**

**Doubly Symmetrical I-Shaped Members, Channels, HSS, and Box Sections Subject to Torsion**

1. **Round and Rectangular HSS, and Box Sections Subject to Torsion**

   The design torsional strength, \( \psi_T T_n \), and the allowable torsional strength, \( T_n / \Omega_T \), for round and rectangular HSS according to the limit states of torsional yielding and torsional buckling shall be determined as follows:

\[ T_n = 0.6 F_v C \psi_T \]  
\[ \psi_T = 0.90 \text{ (LRFD)} \]  
\[ \Omega_T = 1.67 \text{ (ASD)} \]

where

\[ C = \text{HSS torsional constant, in.}^3 \text{ (mm}^3) \]

**User Note:** The torsional constant, \( C \), may be conservatively taken as:

For round HSS:

\[ C = \frac{\pi (D^4 - D^2_t)}{32} \]

For rectangular HSS:

\[ C = 2(B - t)H^2 t - 4.5(4 - \pi) t^3 \]

**1. Round HSS**

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

(a) When \( \frac{L}{D} \leq 4.21 \sqrt[3]{\lambda} \)

\[ C_v = C_{vM} \]  
\[ \text{(G8-2)} \]

(b) When \( \frac{L}{D} > 4.21 \sqrt[3]{\lambda} \)

\[ C_v = C_{vL} \]  
\[ \text{(G8-3)} \]
where

\[ \lambda = \frac{D}{t} \]

\[ D = \text{outside diameter, in. (mm)} \]

\[ L = \text{length of member, in. (mm)} \]

\[ t = \text{design thickness, as defined in Section B4.2, in. (mm)} \]

\[ \frac{0.80}{L/D} \leq \lambda \leq \frac{0.80}{L/D} \]

(1) \( C_{vM1} \) is determined as follows:

(i) When \( \lambda \leq \frac{0.80}{L/D} \)

\[ C_{vM1} = 1.0 \quad \text{(G82-42)} \]

(ii) When \( \frac{0.80}{L/D} < \lambda \leq \frac{3.77}{L/D} \)

\[ C_{vM1} = 1.0 - 0.61 \left( 0.54 \sqrt{\frac{L/D}{E/F_y}} \lambda^{2.35} \right) \]

(1.25 \( y \))

(2) \( C_{vL2} \) is determined as follows:

(i) When \( \lambda \leq \frac{0.20}{L/D} \)

\[ C_{vL2} = 1.0 \quad \text{(G82-75)} \]

(ii) When \( \frac{0.20}{L/D} < \lambda \leq \frac{1.87}{L/D} \)

\[ C_{vL2} = 2.05 \frac{E}{\sqrt{L/D(D/t)^{1.25}F_y}} \]

\[ \lambda^{1.25} \frac{E}{L/D} \]

\[ (G82-64) \]
2. Rectangular HSS and Box Sections

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

(b) For rectangular HSS and box sections

(a1) When $0.74 \leq \frac{h}{t} \leq 0.74 \sqrt{E / F_y}$

$$C_v = 1.2 \quad (G87-108)$$

(b2) When $0.74 \sqrt{E / F_y} < h / t \leq 2.17 \sqrt{E / F_y}$

$$C_v = 1.2 - 0.28 \left( \frac{h}{t} - 0.74 \right) - 1.2 - 0.28 \left( \frac{\lambda}{\sqrt{E / F_y}} - 0.74 \right) \quad (G82-119)$$

(b4) When $2.17 \sqrt{E / F_y} < h / t \leq 5.99 \sqrt{E / F_y}$

$$C_v = \frac{11.23 \sqrt{E / F_y} - h / t}{3.62 \sqrt{E / F_y} + 3.55 h / t} = \frac{11.23 \sqrt{E / F_y} - \lambda}{3.62 \sqrt{E / F_y} + 3.55 \lambda} \quad (G82-120)$$

(c3) When $h / t > 5.99 \sqrt{E / F_y}$

$$C_v = \frac{7.53 E}{(h / t)^{1.5} F_y} \quad (G82-134)$$
where

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

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

\[ \lambda = \frac{h}{t} \]

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

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

For round HSS:

\[ C = \frac{\pi(D^4 - D_0^4)}{32D/2} = \frac{\pi(D - t)^2 t}{2} \]

For rectangular HSS:

\[ C = \frac{2(B - t)(H - t) - 4.5(D - t)^{1.67}}{t} \]

**2. Doubly Symmetric I-Shaped Members and Channels Subject to Torsion**

The available torsional strength for 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:

\[ \phi_T = 0.90 \text{ (LRFD)} \quad \Omega_T = 1.67 \text{ (ASD)} \]

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

\[ F_e = F_T \text{ (G97-12)} \]

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

\[ F_e = 0.6F_T \text{ (G97-244)} \]

(c) For the limit state of buckling

\[ F_e = F_{cr} \text{ (G97-314)} \]

where

\[ F_{cr} = \text{buckling stress for the section as determined by analysis, ksi (MPa)} \]
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:

H1. Doubly Symmetric I-Shaped Members, Channels, HSS, and Box Sections Subject to Flexure and Axial Force

H2. Doubly Symmetric I-Shaped Members, Channels, HSS, and Box Sections Subject to Combined Torsion, Flexure, Shear, and/or Axial Force

H3. Rupture of Flanges with Holes Subjected to Tension

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.

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 \( \frac{P_c}{P_r} \geq 0.2 \)

\[
P_c + \frac{8}{9} \left( \frac{M_{cx}}{M_{cx}} + \frac{M_{cy}}{M_{cy}} \right) \leq 1.0
\]

(H1-1a)

(b) When \( \frac{P_c}{P_r} < 0.2 \)

\[
\frac{P_c}{2P_c} + \left( \frac{M_{cx}}{M_{cx}} + \frac{M_{cy}}{M_{cy}} \right) \leq 1.0
\]

(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, \( \phi P_c \) or \( P_c / \Omega \), determined in accordance with Chapter E, kips (N)

\( M_r \) = required flexural strength, \( \phi M_r = M_r / \Omega \), determined in accordance with Chapter C, using LRFD or ASD load combinations, kip-in. (N-mm)
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

\[ P_r = \text{required tensile strength, determined in accordance with Chapter C, using LRFD or ASD load combinations, kips (N)} \]
\[ P_c = \text{available tensile strength, } \phi P_r \text{ or } P_r / \Omega, \text{ determined in accordance with Chapter D, kips (N)} \]

For doubly symmetric members, \( C_b \) in Chapter F is permitted to be multiplied by \( 1 + \alpha \frac{P_r}{P_c} \) when axial tension acts concurrently with flexure,

where

\[ P_{cy} = \frac{\pi^2 EI_y}{L_b^2} \quad (\text{H1-2}) \]

\[ \alpha = 1.0 \text{ (LRFD); } \alpha = 1.6 \text{ (ASD)} \]

and

\[ E = \text{modulus of elasticity of stainless steel} \]
\[ = 28,000 \text{ ksi (193 000 MPa) for austenitic, and 29,000 ksi (200 000 MPa) for duplex stainless steel} \]
\[ I_y = \text{moment of inertia about the } y\text{-axis, in.}^4 \text{ (mm}^4\text{)} \]
\[ L_b = \text{length between points that are either braced against lateral displacement of the compression flange or braced against twist of the cross section, in. (mm)} \]

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

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\[
\frac{P_c}{P_e} + \frac{M_c}{M_e} + \left(\frac{V_c}{V_e} + \frac{T_c}{T_e}\right)^2 \leq 1.0 \quad \text{(H2-1)}
\]

where

- \( P_c \) = required axial strength, determined in accordance with Chapter C, using LRFD or ASD load combinations, kips (N)
- \( P_e \) = available tensile or compressive strength, \( \phi P_a \) or \( P_a / \Omega \), determined in accordance with Chapter D or E, kips (N)
- \( M_c \) = required flexural strength, determined in accordance with Chapter C, using LRFD or ASD load combinations, kip-in. (N-mm)
- \( M_e \) = available flexural strength, \( \phi M_a \) or \( M_a / \Omega \), determined in accordance with Chapter F, kip-in. (N-mm)
- \( V_c \) = required shear strength, determined in accordance with Chapter C, using LRFD or ASD load combinations, kips (N)
- \( V_e \) = available shear strength, \( \phi V_a \) or \( V_a / \Omega \), determined in accordance with Chapter G, kips (N)
- \( T_c \) = required torsional strength, determined in accordance with Chapter C, using LRFD or ASD load combinations, kip-in. (N-mm)
- \( T_e \) = available torsional strength, \( \phi T_a \) or \( T_a / \Omega \), determined in accordance with Section G7.1, kip-in. (N-mm)

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

The available strength for 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 G7.2 for the limit states of yielding under normal stress, shear yielding under shear stress, or buckling.

H3. RUPTURE OF FLANGES WITH HOLES SUBJECTED TO TENSION

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

\[
\frac{P_c}{P_e} + \frac{M_{cx}}{M_{ex}} \leq 1.0 \quad \text{(H3-1)}
\]

where

- \( P_c \) = 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)
- \( P_e \) = available axial strength for the limit state of tensile rupture of the net section at the location of bolt holes, \( \phi P_a \) or \( P_a / \Omega \), determined in accordance with Section D2(b), kips (N)
- \( M_{cx} \) = 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)
\( M_{cf} \) = available flexural strength about x-axis for the limit state of tensile rupture of the flange, \( \phi M_n \) or \( M_n / \Omega \), 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 composed of structural stainless steel shapes, or HSS, and structural concrete acting together, and structural stainless steel beams supporting a reinforced concrete slab so interconnected that the beams and the slab act together to resist bending 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.

The chapter is organized as follows:

   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, $\phi R_n$, and the allowable strength, $R_n/\Omega$, 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 A3.1d, by complete-joint-penetration (CJP) groove welds, the following provisions apply: (a) material notch-toughness requirements as given in Sections A3.1d; (b) weld access hole details as given in Section J1.6; (c) filler metal requirements as given in Section J2.6; and (d) thermal cut surface preparation and inspection requirements as given in Section M2.4. 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 access holes shall be adequate for deposition of sound weld metal and provide clearance for weld tabs.

(c) The 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 1.5 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) where practical. The reentrant corner is permitted to be formed by mechanical cutting. Thermal cutting by plasma or laser is also permitted if at least 1/8 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 access hole and weld access holes shall be free of notches and sharp reentrant corners.

(f) If dimensions of the part allow, the weld access hole should have a radius not less than 3/8 in. (10 mm).

(g) The access hole is permitted to terminate perpendicular to the flange, providing the weld is terminated at least a distance equal to the weld size away from the access hole.

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 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 acceptable.

User Note: Details of welds for stainless steel are generally similar to details of welds for carbon-steel alloys. Further information is given in AISC Design Guide 27.

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. 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 corrosion testing of duplex stainless steel welds is required, Standard Test Method ASTM A1084 is used for the lean duplex steels UNS S32101, S32202, and S32304. Standard Test Method ASTM A923 is used for the higher alloyed duplex steels S32003, S32205, S32760, S32750, and S82441. The user note on corrosion testing of duplex stainless steels in Section A3.1b (h) also applies.

**User Note:** Laser and Laser-Hybrid shapes produced in accordance with ASTM A1069 are manufactured from AWS D1.6/1.6M approved base metals.

**User Note:** The need for back-side shielding of welds should be considered.

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 non-tubular austenitic stainless steel 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 1/8 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 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 full penetration 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.

<table>
<thead>
<tr>
<th>Welding Process</th>
<th>Welding Position</th>
<th>Groove Type (AWS D1.6, Figure 5.3)</th>
<th>Effective Throat</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shielded metal arc (SMAW)</td>
<td>All</td>
<td>J or U groove</td>
<td>depth of groove</td>
</tr>
<tr>
<td>Gas metal arc (GMAW)</td>
<td>All</td>
<td>J or U groove</td>
<td>depth of groove</td>
</tr>
<tr>
<td>Flux cored arc (FCAW)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Submerged arc (SAW)</td>
<td>F</td>
<td>J or U groove</td>
<td>depth of groove</td>
</tr>
<tr>
<td>Gas metal arc (GMAW)</td>
<td>F, H</td>
<td>45° bevel</td>
<td>depth of groove</td>
</tr>
<tr>
<td>Flux cored arc (FCAW)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shielded metal arc (SMAW)</td>
<td>All</td>
<td>45° bevel</td>
<td>depth of groove minus 1/8 in. (3 mm)</td>
</tr>
<tr>
<td>Gas metal arc (GMAW)</td>
<td>V, OH</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flux cored arc (FCAW)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

---

**Table J2.2**

Effective Throat of Flare-Groove Welds

<table>
<thead>
<tr>
<th>Flare-Bevel-Groove Welds</th>
<th>Flare-V-Groove Welds</th>
</tr>
</thead>
<tbody>
<tr>
<td>(5/16)R</td>
<td>(1/2)R²</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Material Thickness of Thinner Part Joined, in. (mm)</th>
<th>Minimum Effective Throat,(^{[a]}) in. (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>To 1/4 (6) inclusive</td>
<td>1/8 (3)</td>
</tr>
<tr>
<td>Over 1/4 (6) to 1/2 (13)</td>
<td>3/16 (5)</td>
</tr>
<tr>
<td>Over 1/2 (13) to 3/4 (19)</td>
<td>1/4 (6)</td>
</tr>
<tr>
<td>Over 3/4 (19) to 1-1/2 (38)</td>
<td>5/16 (8)</td>
</tr>
<tr>
<td>Over 1-1/2 (38) to 2-1/4 (57)</td>
<td>3/8 (10)</td>
</tr>
<tr>
<td>Over 2-1/4 (57) to 6 (150)</td>
<td>1/2 (13)</td>
</tr>
<tr>
<td>Over 6 (150)</td>
<td>5/8 (16)</td>
</tr>
</tbody>
</table>

\(^{[a]}\) See Table J2.1.

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 not be less than the size required to transmit calculated forces, nor the size as shown in Table J2.43. 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 1/4 in. (6 mm) thick; not greater than the thickness of the material.

2) Along edges of material 1/4 in. (6 mm) or more in thickness; not

TABLE J2.43
Minimum Size of Fillet Welds

<table>
<thead>
<tr>
<th>Material Thickness of Thinner Part Joined, in. (mm)</th>
<th>Minimum Size of Fillet Weld,(^{[a]}) in. (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>To 1/4 (6) inclusive</td>
<td>1/8 (3)</td>
</tr>
<tr>
<td>Over 1/4 (6) to 1/2 (13)</td>
<td>3/16 (5)</td>
</tr>
<tr>
<td>Over 1/2 (13) to 3/4 (19)</td>
<td>1/4 (6)</td>
</tr>
<tr>
<td>Over 3/4 (19)</td>
<td>5/16 (8)</td>
</tr>
</tbody>
</table>

\(^{[a]}\) Leg dimension of fillet welds. Single pass welds must be used.

Note: See Section J2.2b for maximum size of fillet welds.

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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.

(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 mini-
um length of an intermittent fillet weld segment shall be 1-1/2 in [40
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, \( \beta \), determined as:

\[
\beta = 1.2 - 0.002\left(\frac{l}{w}\right) \leq 1.0
\]

where

\[
l = \text{actual length of end-loaded weld, in. (mm)}
\]

\[
w = \text{size of weld leg, in. (mm)}
\]

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 1-1/2 in. (40
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 lat-
teral 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 longi-
tudinal 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 par-
allel to the faying surface shall not terminate at corners of parts or mem-
bers, 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.

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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 \( \frac{3}{8} \) in. thick or less are stopped 4 to 6 times the web thickness from the web toe of the flange-to-web fillet weld, except where the end of the stiffener is welded to the flange.

Details of fillet weld terminations may be shown on shop standard details.

(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 which 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 2-1/4 times the thickness of the weld.
(b) The minimum center-to-center spacing of plug welds shall be four times the diameter of the hole.
(c) The 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 2-1/2 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 2-1/4 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, $\phi R_n$ and the allowable strength, $R_n/\Omega$, 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

$$R_n = F_{nBM} A_{BM}$$  \hspace{1cm} (J2-2)

For the weld metal:

$$R_n = F_{nwA_{we}}$$  \hspace{1cm} (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 $\phi$, $\Omega$, $F_{nBM}$ and $F_{nw}$, and limitations thereon, are given in Table J2.54.

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. For material that is cold worked, the welded joint will be annealed and therefore lower strength than the base metal. However, the strength of the base metal in the heat affected zones should be taken as the tensile strength of the annealed base metal.
### TABLE J2.54
Available Strength of Welded Joints—Except Laser and Laser Hybrid And Filler Metals With Matching Strength, ksi (MPa)\[c, d\]

<table>
<thead>
<tr>
<th>Load Type and Direction Relative to Weld Axis</th>
<th>Pertinent Metal</th>
<th>(\phi) and (\Omega)</th>
<th>Nominal Stress ((F_{\text{nom}}) or (F_{\text{w}}), ksi (MPa))</th>
<th>Effective Area ((A_{\text{nom}}) or (A_{\text{w}}), in.(^2) (mm(^2)))</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>COMPLETE-JOINT-PENETRATION GROOVE WELDS</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tension—Normal to weld axis</td>
<td>Base</td>
<td>(\phi = 0.75)</td>
<td>(\Omega = 2.00)</td>
<td>(F_{\text{XXF}}) [a]</td>
</tr>
<tr>
<td>Compression—Normal to weld axis</td>
<td>Base</td>
<td>(\phi = 0.90)</td>
<td>(\Omega = 1.67)</td>
<td>(F_{\gamma})</td>
</tr>
<tr>
<td>Weld</td>
<td>(\phi = 0.75)</td>
<td>(\Omega = 2.00)</td>
<td>(F_{\text{XXF}}) [a]</td>
<td>See J2.1a</td>
</tr>
<tr>
<td><strong>Tension or compression—Parallel to weld axis</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear</td>
<td>Base</td>
<td>(\phi = 0.75)</td>
<td>(\Omega = 2.00)</td>
<td>(0.60 F_{\text{XXF}}) [b]</td>
</tr>
<tr>
<td><strong>PARTIAL-JOINT-PENETRATION GROOVE WELDS INCLUDING FLARE V-GROOVE AND FLARE BEVEL GROOVE WELDS</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tension—Normal to weld axis</td>
<td>Base</td>
<td>(\phi = 0.75)</td>
<td>(\Omega = 2.00)</td>
<td>(F_{u})</td>
</tr>
<tr>
<td>Weld</td>
<td>(\phi = 0.80)</td>
<td>(\Omega = 1.88)</td>
<td>(0.60 F_{\text{XXF}}) [a]</td>
<td>See J2.1a</td>
</tr>
<tr>
<td>Compression—Column to base plate and column splices designed per Section J1.4(a)</td>
<td></td>
<td></td>
<td></td>
<td>Compressive stress is permitted to be neglected in design of welds joining the parts.</td>
</tr>
<tr>
<td>Compression—Connections of members designed to bear other than columns as described in Section J1.4(b)</td>
<td>Base</td>
<td>(\phi = 0.90)</td>
<td>(\Omega = 1.67)</td>
<td>(F_{\gamma})</td>
</tr>
<tr>
<td>Weld</td>
<td>(\phi = 0.80)</td>
<td>(\Omega = 1.88)</td>
<td>(0.60 F_{\text{XXF}}) [a]</td>
<td>See J2.1a</td>
</tr>
<tr>
<td>Compression—Connections not finished-to-bear Connections</td>
<td>Base</td>
<td>(\phi = 0.90)</td>
<td>(\Omega = 1.67)</td>
<td>(F_{\gamma})</td>
</tr>
<tr>
<td>Weld</td>
<td>(\phi = 0.80)</td>
<td>(\Omega = 1.88)</td>
<td>(0.90 F_{\text{XXF}}) [a]</td>
<td>See J2.1a</td>
</tr>
<tr>
<td>Tension or compression—Parallel to weld axis</td>
<td>Shear</td>
<td>Base</td>
<td>(\phi = 0.75)</td>
<td>(\Omega = 2.00)</td>
</tr>
</tbody>
</table>
FILLET WELDS INCLUDING FILLETS IN HOLES AND SLOTS AND SKEWED T-JOINTS

<table>
<thead>
<tr>
<th>Shear</th>
<th>Base</th>
<th>Governed by J4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weld</td>
<td>φ = 0.75</td>
<td>0.60 ( F_{EXX} ) [a] [b]</td>
</tr>
<tr>
<td>Tension or compression—Parallel to weld axis</td>
<td>Tension or compression in parts joined parallel to a weld is permitted to be neglected in design of welds joining the parts.</td>
<td></td>
</tr>
</tbody>
</table>

PLUG AND SLOT WELDS

<table>
<thead>
<tr>
<th>Shear—Parallel to faying surface on the effective area</th>
<th>Base</th>
<th>Governed by J4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weld</td>
<td>φ = 0.75</td>
<td>0.60 ( F_{EXX} ) [a]</td>
</tr>
</tbody>
</table>

[a] 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, as follows:

- (1) For covered electrodes, nominal tensile strength shall be that required in AWS A5.4/A5.4M.
- (2) For flux cored and metal cored filler metals, nominal tensile strength shall be that required in AWS A5.22/A5.22M.
- (3) For solid filler metals, nominal tensile strength shall be that required in AWS A5.4/A5.4M for covered electrodes of corresponding composition of weld metal.
- (4) For filler metals not covered in AWS A5.4/A5.4M, AWS A5.9/A5.9M, or A5.22/A5.22M, nominal tensile strength shall be assessed by the engineer.

[b] The provisions of Section J2.4(b) are also applicable.
[c] The strength of laser and laser hybrid welds is equivalent to the base metal.
[d] 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.

(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,

\[
\phi = 0.75 \text{ (LRFD)}; \quad \Omega = 2.00 \text{ (ASD)}
\]

\[\theta = \text{angle between the line of action of the required force and the weld longitudinal axis, degrees}\]

(1) For a linear weld group with a uniform leg size, loaded through the center of gravity

\[
R_o = F_{uw} A_w \quad \text{(J2-4)}
\]

where

\[
F_{uw} = 0.60 F_{EXX} \left(1.0 + 0.50 \sin 1.5\theta\right), \text{ ksi (MPa)} \quad \text{(J2-5)}
\]

\[
F_{EXX} = \text{filler metal classification strength, ksi (MPa)}
\]

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

(2) For fillet weld groups concentrically loaded and consisting of elements with a uniform leg size that are oriented both longitudinally

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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:

\[
\begin{align*}
(i) \quad R_n &= R_{nwl} + R_{nwt} \\
(ii) \quad R_n &= 0.85 R_{nwl} + 1.5 R_{nwt}
\end{align*}
\]

where \( R_{nwl} \) = total nominal strength of longitudinally loaded fillet welds, as determined in accordance with Table J2.54, kips (N)

\( R_{nwt} \) = total nominal strength of transversely loaded fillet welds, as determined in accordance with Table J2.54 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 be over-alloyed relative to the base metal in order to obtain weld corrosion resistance equivalent to the base metal. The corrosion resistance of the filler metal is as important as the strength.

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.

User Note: 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 weld shall be at least as high as the specified tensile strength of the filler metal. The EOR shall specify the filler metal to ensure adequate corrosion resistance and strength.

User Note: AISC Design Guide 27 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 PWS 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.
The Engineer should specify the filler metal to ensure adequate corrosion resistance and strength.

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: Annex D of AWS D1.6/1.6M gives suggested filler metals for welding austenitic stainless steels to themselves or to other steels. When welding stainless steels to carbon 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 involving other steel alloys and stainless steels, it is generally recommended that any paint system applied to the other carbon steel alloy should extend over the bimetallic weldment onto the stainless steel to a distance of at least 2 in. (20 mm).

Any coating system applied to the other carbon steel alloy 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 Specification for Structural Joints Using High-Strength Bolts, hereafter referred to as the RCSC Specification, as approved by the Research Council on Structural Connections:

- 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, Table C-2.1, Table C-2.2, and 2.4.2

Stainless steel bolts intended for slip-critical and fully tightened 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 Section 2.8 of the RCSC Specification, are permitted if they meet the materials, manufacturing, chemical composition requirements and mechanical property requirements of the ASTM specification standards listed in Section A3.3 of this specification.
• SECTION 3. BOLTED PARTS: Section 3.2.1 (uncoated faying surfaces only), Section 3.2.2 (1) (uncoated faying surfaces only), Section 3.3 and Section 3.4.

**User Note:** Section 3.1 does not apply. Compressible materials placed within the grip of the bolt are permitted for snug tightened connections between dissimilar metals to provide electrical insulation to prevent galvanic and crevice corrosion (see Section J3.12).

**User Note:** The faying surface of slip-critical joints shall be as rolled, shot blasted or grit blasted. If the faying surface is as rolled, then no special surface preparation is required.

• SECTION 4. JOINT TYPE

• SECTION 7. PRE-INSTALLATION VERIFICATION, except the minimum bolt pretension for pre-installation verification is given as $1.054 T_b$ where $T_b$ is given in Equation J3-5 and not by Table 7.1 of the RCSC Specification.

**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.

• SECTION 8. INSTALLATION: Section 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. Suitability for the preloading of a bolting assembly shall be certified by procedure testing. Twist-off-type tension-control bolt pretensioning and direct tension-indicator pretensioning are outside the scope of this Specification.

**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.

• 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.

**User Note:** Refer to the Commentary for further information about the scope of each stainless steel bolt ASTM standard.

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.

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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 carbon 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., UNS 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 as smooth as possible 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 subjected 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 on 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 1/4 in. (6 mm) are permitted in slip-critical connections
designed on the basis of standard holes without reducing the nominal shear
strength of the fastener to that specified for slotted holes.

(d) Oversized holes are permitted in any or all plies of slip-critical connec-
tions, 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 bear-
ing-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 ei-
ther a slip-critical or bearing-type connection at an individual faying sur-
face. 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 un-
der both the bolt head and nut in pretensioned connections subject to fa-
tigue loading, and slip-critical joints.

<table>
<thead>
<tr>
<th>Bolt Diameter</th>
<th>Standard Hole Dimensions</th>
<th>Oversize Hole Dimensions</th>
<th>Short-Slot Hole Dimensions (Width x Length)</th>
<th>Long-Slot Hole Dimensions (Width x Length)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/2</td>
<td>9/16</td>
<td>5/8</td>
<td>9/16 x 11/16</td>
<td>9/16 x 1-1/4</td>
</tr>
<tr>
<td>5/8</td>
<td>-11/16</td>
<td>13/16</td>
<td>11/16 x 7/8</td>
<td>11/16 x 1-9/16</td>
</tr>
<tr>
<td>3/4</td>
<td>13/16</td>
<td>15/16</td>
<td>15/16 x 1-1/8</td>
<td>15/16 x 2-3/16</td>
</tr>
<tr>
<td>7/8</td>
<td>15/16</td>
<td>1-1/16</td>
<td>1-1/8 x 1-5/16</td>
<td>1-1/8 x 2-1/2</td>
</tr>
<tr>
<td>1</td>
<td>1-1/8</td>
<td>1-1/8</td>
<td>(d + 1/8) x (d + 3/8)</td>
<td>(d + 1/8) x 2.5d</td>
</tr>
<tr>
<td>≥1-1/8</td>
<td>d + 1/8</td>
<td>d + 5/16</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Bolt Diameter</th>
<th>Standard Hole Dimensions</th>
<th>Oversize Hole Dimensions</th>
<th>Short-Slot Hole Dimensions (Width x Length)</th>
<th>Long-Slot Hole Dimensions (Width x Length)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M16</td>
<td>18</td>
<td>20</td>
<td>18 x 22</td>
<td>18 x 40</td>
</tr>
<tr>
<td>M20</td>
<td>22</td>
<td>24</td>
<td>22 x 26</td>
<td>22 x 50</td>
</tr>
<tr>
<td>M22</td>
<td>24</td>
<td>28</td>
<td>24 x 30</td>
<td>24 x 55</td>
</tr>
<tr>
<td>M24</td>
<td>27[a]</td>
<td>30</td>
<td>27 x 32</td>
<td>27 x 60</td>
</tr>
<tr>
<td>M27</td>
<td>30</td>
<td>35</td>
<td>30 x 37</td>
<td>30 x 67</td>
</tr>
<tr>
<td>M30</td>
<td>33</td>
<td>38</td>
<td>33 x 40</td>
<td>33 x 75</td>
</tr>
<tr>
<td>≥M36</td>
<td>d + 3</td>
<td>d + 8</td>
<td>(d + 3) x (d + 10)</td>
<td>(d + 3) x 2.5d</td>
</tr>
</tbody>
</table>

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

3. Minimum Spacing
The distance between centers of standard, oversized or slotted holes shall not be less than 2-2/3 times the nominal diameter, \( d \), of the fastener. However, the clear distance between bolt holes or slots shall not be less than \( d \).

**User Note:** A distance between centers of standard, oversize or slotted holes of 3\( d \) 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).

**TABLE J3.2**

<table>
<thead>
<tr>
<th>Bolt Diameter</th>
<th>Minimum Edge Distance</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \frac{1}{2} )</td>
<td>( \frac{3}{4} )</td>
</tr>
<tr>
<td>( \frac{5}{8} )</td>
<td>( \frac{7}{8} )</td>
</tr>
<tr>
<td>( \frac{3}{4} )</td>
<td>1</td>
</tr>
<tr>
<td>( \frac{7}{8} )</td>
<td>1-1/8</td>
</tr>
<tr>
<td>1</td>
<td>1-1/4</td>
</tr>
<tr>
<td>1-1/8</td>
<td>1-1/2</td>
</tr>
<tr>
<td>1-1/4</td>
<td>1-5/8</td>
</tr>
<tr>
<td>Over 1-1/4</td>
<td>1-1/4( d )</td>
</tr>
</tbody>
</table>

*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.*

*For oversized or slotted holes, see Table J3.3.*
TABLE J3.2M
Minimum Edge Distance\(^\text{[a]}\) from Center of Standard Hole\(^\text{[b]}\) to Edge of Connected Part, mm

<table>
<thead>
<tr>
<th>Bolt Diameter</th>
<th>Minimum Edge Distance</th>
</tr>
</thead>
<tbody>
<tr>
<td>16</td>
<td>22</td>
</tr>
<tr>
<td>20</td>
<td>26</td>
</tr>
<tr>
<td>22</td>
<td>28</td>
</tr>
<tr>
<td>24</td>
<td>30</td>
</tr>
<tr>
<td>27</td>
<td>34</td>
</tr>
<tr>
<td>30</td>
<td>38</td>
</tr>
<tr>
<td>36</td>
<td>46</td>
</tr>
<tr>
<td>Over 36</td>
<td>1.25(d)</td>
</tr>
</tbody>
</table>

\(^{[a]}\) 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>
<thead>
<tr>
<th>Nominal Diameter of Fastener</th>
<th>Oversized Holes</th>
<th>Slotted Holes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Long Axis Perpendicular to Edge</td>
<td>Long Axis Parallel to Edge</td>
</tr>
<tr>
<td></td>
<td>Short Slots</td>
<td>Long Slots(^{[a]})</td>
</tr>
<tr>
<td>(\leq 7/8)</td>
<td>1/16</td>
<td>3/4(d)</td>
</tr>
<tr>
<td>1</td>
<td>1/8</td>
<td>0</td>
</tr>
<tr>
<td>(\geq 1 1/8)</td>
<td>1/8</td>
<td>3/16</td>
</tr>
</tbody>
</table>

\(^{[a]}\) When the length of the slot is less than the maximum allowable (see Table J3.1), 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

<table>
<thead>
<tr>
<th>Nominal Diameter of Fastener</th>
<th>Oversized Holes</th>
<th>Slotted Holes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Long Axis Perpendicular to Edge</td>
<td>Long Axis Parallel to Edge</td>
</tr>
<tr>
<td></td>
<td>Short Slots</td>
<td>Long Slots(^{[a]})</td>
</tr>
<tr>
<td>(\leq 22)</td>
<td>2</td>
<td>3 (0.75d)</td>
</tr>
<tr>
<td>24</td>
<td>3</td>
<td>5</td>
</tr>
<tr>
<td>(\geq 27)</td>
<td>3</td>
<td>5</td>
</tr>
</tbody>
</table>

\(^{[a]}\) When the length of the slot is less than the maximum allowable (see Table J3.1M), is permitted to be reduced by one-half the difference between the maximum and actual slot lengths.

6. Tensile and Shear Strength of Bolts and Threaded Parts

The design tensile or shear strength, \(\phi R_n\), and the allowable tensile or shear strength, \(R_n/\Omega\), of a snug tightened bolt or pretensioned bolt or threaded part shall be determined according to the limit states of tension rupture and shear rupture as:

\[
R_n = F_n A_b \tag{J3-1}
\]

\[
\phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)}
\]

where

\(A_b\) = nominal unthreaded body area of bolt or threaded part, in.\(^2\) (mm\(^2\))

\(F_n\) = nominal tensile stress, \(F_n\), or shear stress, \(F_n\), ksi (MPa)

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\[ F_{nt} = 0.75 F_u \]
\[ F_{nv} = 0.45 F_u \] if threads are not excluded from the shear planes
\[ F_{nt} = 0.55 F_u \] if threads are excluded from the shear planes

The value for \( F_u \) should 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 and with dimensions in accordance with ASME B18.2.6.

The above design rules can also be applied to precipitation hardening fasteners in accordance with ASTM F593 with the following resistance and safety factors:

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 subjected to combined tension and shear shall be determined according to the limit states of tension and shear rupture as:

\[ R_n = F'_{nt} A_b \]  

where

\[ F'_{nt} = \text{nominal tensile stress modified to include the effects of shear stress, ksi (MPa)} \]

\[ F_{nt} = 1.3 F_{nt} - \frac{F_{nt}}{\phi_{F_{nv}}} f_{rv} \leq F_{nt} \] (LRFD)  

\[ F_{nt} = 1.3 F_{nt} - \frac{\Omega F_{nt}}{F_{nv}} f_{rv} \leq F_{nt} \] (ASD)  

\[ F_{nt} = \text{nominal tensile stress from Section J3.6, ksi (MPa)} \]

\[ F_{nv} = \text{nominal shear stress from Section J3.6, ksi (MPa)} \]

\[ f_{rv} = \text{required shear stress using LRFD or ASD load combinations, ksi (MPa)} \]

The available shear stress of the fastener shall equal or exceed the required shear stress, \( f_{rv} \).
The required tensile strength (including any force due to prying action) must also be less than the available tensile strength.

**User Note:** Note that when the required stress, \( f \), in either shear or tension, is less than or equal to 30% of the corresponding available stress, the effects of combined stress need not be investigated. Also note that Equations J3-3a and J3-3b can be rewritten so as to find a nominal shear stress, \( F'_{nv} \), as a function of the required tensile stress, \( f \).

8. **Stainless Steel Bolts in Slip-Critical Connections**

Slip-critical connections shall be designed to prevent slip and for the limit states of bearing-type connections. When slip-critical bolts pass through fillers, all surfaces subject to slip shall be prepared to achieve the design slip resistance.

For bolts used in slip-resistant connections,

\[ 80 \text{ ksi (550 MPa)} \leq F_yb \leq 116 \text{ ksi (800 MPa)} \]

where

\[ F_yb = \text{specified minimum yield strength of bolt, ksi (MPa)} \]

Suitability for preloading of a bolting assembly shall be certified by procedure testing, which takes into account the specific components in the bolting assembly, pretensioning method, and lubrication.

**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. The bolt tightening qualification procedure given in AISC Design Guide 27 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. Guidance on a bolt tightening qualification procedure is given in the Commentary.

The single bolt available slip resistance for the limit state of slip shall be determined as follows:

\[ R_n = \mu D_h h_f T_b n_x \]  \hspace{1cm} (J3-4)

(a) For standard size and short-slotted holes perpendicular to the direction of the load

\[ \phi = 1.00 \text{ (LRFD)} \quad \Omega = 1.50 \text{ (ASD)} \]

(b) For oversized and short-slotted holes parallel to the direction of the load

\[ \phi = 0.85 \text{ (LRFD)} \quad \Omega = 1.76 \text{ (ASD)} \]

(c) For long-slotted holes

\[ \phi = 0.70 \text{ (LRFD)} \quad \Omega = 2.14 \text{ (ASD)} \]

where

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\[ D_v = 1.013, \text{ a multiplier that reflects the ratio of the mean installed bolt pretension to the specified minimum bolt pretension. The use of other values are permitted if approved by the EOR.} \]

\[ T_b = \text{minimum fastener tension for stainless steel bolts, determined as:} \]

\[ T_b = 0.7 F_{bi} A_t \quad (J3-5) \]

\[ A_t = \text{tensile stress area of bolt, in.}^2 \text{ (mm}^2) \]

\[ h_f = \text{factor for fillers, determined as follows:} \]

(1) For one filler between connected parts

\[ h_f = 1.0 \]

(2) For two or more fillers between connected parts

\[ h_f = 0.85 \]

\[ n_s = \text{number of slip planes required to permit the connection to slip} \]

\[ \mu = \text{mean slip coefficient, given in Table J3.4} \]

<table>
<thead>
<tr>
<th>Surface-condition</th>
<th>Slip coefficient ( \mu ) for Friction Surfaces</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class</td>
<td></td>
</tr>
<tr>
<td>Class</td>
<td>Surface finish(^[a])</td>
</tr>
<tr>
<td>Surface as rolled</td>
<td>( \geq 35 )</td>
</tr>
<tr>
<td>Surface blasted with shot(^[b])</td>
<td>( \geq 35 )</td>
</tr>
<tr>
<td>Surface blasted with grit(^[c])</td>
<td>( \geq 45 )</td>
</tr>
<tr>
<td>SSD</td>
<td></td>
</tr>
</tbody>
</table>

\(^{[a]}\) 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. \(^{[c]}\) \( R_z \) is the surface roughness according to ASTM D7127.

\(^{[d]}\) Virgin grit or shot media shall be used to avoid the embedment of tramp iron in the surface of the stainless steel which may lead to corrosion.

\(^{[c]}\) The potential loss of preloading force due to time dependent relaxation from its initial value is considered in these slip coefficient values.

\(^{[d]}\) The slip coefficient for a grit blasted surface in which \( R_z \geq 55 \mu \text{m} \) may be taken as 0.5 if demonstrated by testing in accordance with Appendix A of the RCSC Specification.

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 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 subjected to an applied tension that reduces the net clamping force, the available slip resistance per bolt from Section J3.8 shall be multiplied by the factor, $k_{sc}$, determined as follows:

$$k_{sc} = 1 - \frac{T_a}{D_s T_b n_b} \geq 0 \quad \text{(LRFD)} \quad (J3-6a)$$

$$k_{sc} = 1 - \frac{1.5 T_u}{D_s T_b n_b} \geq 0 \quad \text{(ASD)} \quad (J3-6b)$$

where
- $T_a$ = required tension force using ASD load combinations, kips (kN)
- $T_u$ = required tension force using LRFD load combinations, kips (kN)
- $n_b$ = number of bolts carrying the applied tension

10. Bearing and Tearout Strength at Bolt Holes

The available strength, $\phi R_u$ and $R_u/\Omega$, at bolt holes shall be determined for the limit states of bearing and tearout, as follows:

$$\phi = 0.75 \quad \text{(LRFD)}$$

$$\Omega = 2.00 \quad \text{(ASD)}$$

The nominal strength of the connected material, $R_u$, 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

$$R_u = \alpha_d dF_u \quad (J3-7a)$$

where

$$\alpha_d = 1.25 \left( \frac{e_1}{2 d_h} \right) \leq 1.25 \quad \text{for end bolts} \quad (J3-8a)$$

$$\alpha_d = 1.25 \left( \frac{p_n}{4 d_h} \right) \leq 1.25 \quad \text{for inner bolts} \quad (J3-9a)$$

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

$$R_u = \alpha_t dF_u \quad (J3-7b)$$

where

$$\alpha_t = 2.5 \left( \frac{e_1}{3 d_h} \right) \leq 2.5 \quad \text{for } e_2/d_h > 1.5 \quad \text{for end bolts} \quad (J3-8b)$$

$$\alpha_t = 2.5 \left( \frac{e_1}{3 d_h} \right) \leq 2.0 \quad \text{for } e_2/d_h \leq 1.5 \quad \text{for end bolts} \quad (J3-9b)$$
\[ \alpha_i = 2.5 \left( \frac{P_1}{6d_h} \right) \leq 2.5 \quad \text{for} \quad \frac{P_2}{d_h} > 3.0 \quad \text{for inner bolts} \quad (J3-10b) \]

\[ \alpha_i = 2.5 \left( \frac{P_1}{6d_h} \right) \leq 2.0 \quad \text{for} \quad \frac{P_2}{d_h} \leq 3.0 \quad \text{for inner bolts} \quad (J3-11b) \]

**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

\[ R_u = 1.2l_i F_u \quad (J3-7c) \]

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

\[ R_u = 1.5l_i F_u \quad (J3-7d) \]

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

(1) Bearing

\[ R_u = 0.6 \alpha_i d l_i F_u \quad (J3-7e) \]

(2) Tearout

\[ R_u = 1.0l_i F_u \quad (J3-7f) \]

(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 fastener diameter, in. (mm)
- \( l_i \) = clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material, in. (mm)
- \( e_1 \) = minimum value of the end distance, in. (mm) (See Table J3.5)
- \( e_2 \) = minimum value of the edge distance, in. (mm) (See Table J3.5)
- \( p_1 \) = minimum value of the center-to-center spacing of bolts in the direction of stress, in. (mm) (See Table J3.5)
- \( p_2 \) = minimum value of the center-to-center spacing of bolts normal to the direction of stress, in. (mm) (See Table J3.5)
- \( d_h \) = diameter of hole, 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 carbon steel alloy bolts, including galvanized steel bolts, with stainless steel structural 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 bushing are used to provide corrosion protection in snug-tightened connections, the product used in the bolt grip shall be specified approved 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.

User Note: Further information about bolting dissimilar metals is given in AISC Design Guide 27.
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, $\phi R_n$, and the allowable strength, $R_n/\Omega$, 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

$$R_n = F_e A_e$$  \hspace{1cm} (J4-1)

$$\phi = 0.90 \text{ (LRFD)} \hspace{1cm} \Omega = 1.67 \text{ (ASD)}$$

(b) For tensile rupture of connecting elements

$$R_n = F_u A_e$$  \hspace{1cm} (J4-2)

$$\phi = 0.75 \text{ (LRFD)} \hspace{1cm} \Omega = 2.00 \text{ (ASD)}$$

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

$$R_n = 0.60 F_s A_{gs}$$  \hspace{1cm} (J4-3)

$$\phi = 0.90 \text{ (LRFD)} \hspace{1cm} \Omega = 1.67 \text{ (ASD)}$$

where

$A_{gs} =$ gross area subject to shear, in.$^2$ (mm$^2$)

(b) For shear rupture of the element

$$R_n = 0.60 F_u A_{ws}$$  \hspace{1cm} (J4-4)
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:

$$ R_u = 0.60F_y A_{nt} + U_{bs} F_u A_{nt} \leq 0.60F_y A_{nt} + U_{bs} F_u A_{nt} $$  \( (J4-5) \)

where

$$ A_{nt} = \text{net area subject to tension, in.}^2 (\text{mm}^2) $$

Where the tension stress is uniform, \( U_{bs} = 1 \); where the tension stress is non-uniform, \( U_{bs} = 0.5 \).

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 as follows:

(a) When \( L_c/r \leq 25 \)

$$ P_u = F_y A_{nt} \phi = F_y A_{nt} \text{, } \phi = 0.90 \text{ (LRFD) } \Omega = 1.67 \text{ (ASD) } $$  \( (J4-6) \)

(b) When \( L_c/r > 25 \), the provisions of Chapter E apply;

where

$$ L_c = KL \text{ = effective length of connecting element, in. (mm) } $$

$$ K \text{ = effective length factor } $$

$$ L' = \text{laterally unbraced length of connecting element the member, in. (mm) } $$

User Note: The effective length factors used in computing compressive strengths of connecting elements are specific to the end restraint provided and may not necessarily be taken as unity when the direct analysis method is employed.

5. Strength of Elements in Flexure

The available flexural strength of affected elements shall be the lower value obtained according to the limit states of flexural yielding, local buckling, flexural lateral-torsional buckling, and flexural rupture.
J5. FILLERS

1. 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 1/4 in. (6 mm) thick shall not be used to transfer stress. When the thickness of the fillers is less than 1/4 in. (6 mm), or when the thickness of the filler is 1/4 in. (6 mm) or greater but not sufficient to transfer the applied force between the connected parts, the filler shall be kept flush with the edge of the outside connected part, and the size of the weld shall be increased over the required size by an amount equal to the thickness of the filler.

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 subjected to the applied force in the filler shall be sufficient to prevent overstressing the filler. The welds joining the filler to the inside connected base metal shall be sufficient to transmit the applied force.

3. Fillers in Bolted Bearing-Type Connections

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

(a) The shear strength of the bolts shall be multiplied by the factor

\[ 1 - 0.4(t - 0.25) \]

but not less than 0.85, where \( t \) is the total thickness of the fillers.

(b) The fillers shall be welded or extended beyond the joint and bolted to uniformly distribute the total force in the connected element over the combined cross section of the connected element and the fillers.

(c) The size of the joint shall be increased to accommodate a number of bolts that is equivalent to the total number required in (b).
**J6. SPLICES**

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

**J7. BEARING STRENGTH**

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

\[
\phi = 0.75 \quad \Omega = 2.00
\]

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:

\[
R_n = 1.8 F_y A_{pb}
\]

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 which are used in pin-connected members, as addressed in Section D5.

1. **Bearing Strength**

The design bearing strength, \( \phi R_n \), and the allowable bearing strength, \( R_n/\Omega \), of pins shall be determined in accordance with Section J7.

2. **Shear Strength**

The design shear strength, \( \phi R_n \), and the allowable shear strength, \( R_n/\Omega \), of pins shall be determined as follows:

\[
R_n = 0.6 F_u A_p
\]

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, \( \phi M_n \), and the allowable flexural strength, \( M_n/\Omega \), of pins shall be determined as follows:

\[
M_n = 1.5 S F_y
\]
\[ \phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)} \]

where

\[ S = \text{elastic section modulus of the pin, in.}^2 \text{ (mm}^2 \text{)} \]

\[ F_y = \text{specified minimum yield stress of the pin, ksi (MPa)} \]

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.

\[ \left( \frac{V_r^2}{V_c^2} + \frac{M_r^2}{M_c^2} \right) \leq 1.0 \quad \text{(J8-3)} \]

where

\[ V_r = \text{required shear strength, determined in accordance with Chapter C, using LRFD or ASD load combinations, kips (N)} \]

\[ V_c = \text{available shear strength, } \phi R_y \text{ or } R_y/\Omega, \text{ determined in accordance with Section J8.2, kips (N)} \]

\[ M_r = \text{required flexural strength, } \phi M_u \text{ or } M_u/\Omega, \text{ determined in accordance with Chapter C, using LRFD or ASD load combinations, kip-in. (N-mm)} \]

\[ M_c = \text{available flexural strength, determined in accordance with Section J8.3, kip-in. (N-mm)} \]

19. **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, \( \phi P_p \), and the allowable bearing strength, \( P_p/\Omega \), for the limit state of concrete crushing are permitted to be taken as follows:

\[ \phi_c = 0.65 \text{ (LRFD)} \quad \Omega_c = 2.31 \text{ (ASD)} \]

The nominal bearing strength, \( P_p \), is determined as follows:

(a) On the full area of a concrete support

\[ P_p = 0.85 f'_c A_i \quad \text{(J9-1)} \]

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

\[ P_p = 0.85 f'_c A \sqrt{\frac{A}{A}} \leq 1.7 f'_c A \quad \text{(J9-2)} \]

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, Second Edition_, for column base design information.

When anchor rods are used to resist horizontal forces, hole size, anchor rod setting tolerance, and the horizontal movement of the column shall be considered in the design.

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

**User Note:** There is no separate ASTM standard covering stainless steel anchor rods; they shall 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. See OSHA for special erection requirements for anchor rods.

**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 and similar built-up shapes. A single-concentrated force is either tensile or compressive. Double-concentrated forces are one tensile and one compressive and form a couple on the same side of the loaded member.

When the required strength exceeds the available strength as determined for the limit states listed in this section, stiffeners and/or doublers shall be provided and shall be sized for the difference between the required strength and the available strength for the applicable limit state. Stiffeners shall also meet the design requirements in Section J11.8. Doublers shall also meet the design requirement in Section J11.9.

**User Note:** See Appendix 6, Section 6.3 for requirements for the ends of cantilever members.
Stiffeners are required at unframed ends of beams in accordance with the requirements of Section J11.7.

**User Note:** Design guidance for members other than wide-flange sections and similar built-up shapes can be found in the Commentary.

### 1. Flange Local Bending

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

The design strength, \( \phi R_n \), and the allowable strength, \( \Omega R_n \), for the limit state of flange local bending shall be determined as:

\[
R_n = 6.25 F_{ys} t_f^2
\]  
(J11-1)

\[
\phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)}
\]

where

\( F_{ys} = \text{specified minimum yield stress of the flange, ksi (MPa)} \)

\( t_f = \text{design thickness of the loaded flange, as defined in Section B4.2, in. (mm)} \)

If the length of loading across the member flange is less than 0.15\( b_y \), where \( b_y \) 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 10\( t_f \), \( R_n \) 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:

\[
\phi = 1.00 \text{ (LRFD)} \quad \Omega = 1.50 \text{ (ASD)}
\]

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 depth of the member, \( d \),

\[
R_n = F_{yw} t_w (5k + l_b)
\]  
(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, \( d \),

\[
R_n = F_{yw} t_w (2.5k + l_b)
\]  
(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:

\[
\phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)}
\]

The nominal strength, \( R_n \), shall be determined as follows:

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

\[
R_n = 0.80t_w^2 \left[ 1 + 3 \left( \frac{l_b}{d} \right) \left( \frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{ \frac{EF_{yw}t_f}{t_w} Q_f } \quad \text{(J11-4)}
\]

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

\[
R_n = 0.40t_w^2 \left[ 1 + 3 \left( \frac{l_b}{d} \right) \left( \frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{ \frac{EF_{yw}t_f}{t_w} Q_f } \quad \text{(J11-5a)}
\]

\[
R_n = 0.40t_w^2 \left[ 1 + \left( \frac{4l_b}{d} - 0.2 \right) \left( \frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{ \frac{EF_{yw}t_f}{t_w} Q_f } \quad \text{(J11-5b)}
\]

where

\( d \) = full nominal depth of the member, in. (mm)

\( Q_f = 1.0 \) for wide-flange sections and for HSS (connecting surface) in tension

\( = \) as given in Table K2.2 for all other HSS conditions

When required, a transverse stiffener, a pair of transverse stiffeners, or a doubler plate extending at least three quarters of the depth of the web shall be provided.
4. **Web Sidesway Buckling**

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

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

\[
\phi = 0.85 \text{ (LRFD)} \quad \Omega = 1.76 \text{ (ASD)}
\]

The nominal strength, \( R_n \), shall be determined as follows:

(a) If the compression flange is restrained against rotation

1. When \( \left( \frac{h/t_w}{I_y/b_f} \right) \leq 2.3 \)

\[
R_n = \frac{C_r t_w t_f}{h^2} \left[ 1 + 0.4 \left( \frac{h/t_w}{I_y/b_f} \right)^3 \right]
\]

(J11-6)

2. When \( \left( \frac{h/t_w}{I_y/b_f} \right) > 2.3 \), 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 \( \left( \frac{h/t_w}{I_y/b_f} \right) \leq 1.7 \)

\[
R_n = \frac{C_r t_w t_f}{h^2} \left[ 0.4 \left( \frac{h/t_w}{I_y/b_f} \right)^3 \right]
\]

(J11-7)

2. When \( \left( \frac{h/t_w}{I_y/b_f} \right) > 1.7 \), 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:

\[ C_r = \begin{cases} 
960,000 \text{ ksi (6.6} \times 10^6 \text{ MPa)}, & \text{when } M_o < M_y \text{ (LRFD) or } 1.5M_o < M_y \\
480,000 \text{ ksi (3.3} \times 10^6 \text{ MPa),} & \text{when } M_o \geq M_y \text{ (LRFD) or } 1.5M_o \geq M_y
\end{cases} \]
5. Web Compression Buckling

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

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

\[
R_n = \left( \frac{4\pi^2}{12} \sqrt{EF_y} \right) \frac{Q_f}{h}
\]

(J11-8)

where

\[Q_f = 1.0\] for wide-flange sections and for HSS (connecting surface) in tension.

= as given in Table K3.2 for all other HSS conditions

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

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

6. Web Panel-Zone Shear

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

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

\[\phi = 0.90\] (LRFD) \hspace{1cm} \[\Omega = 1.67\] (ASD)

The nominal strength, \(R_n\), shall be determined as follows:
(a) When the effect of inelastic panel-zone deformation on frame stability is not accounted for in the analysis:

1. For \( \alpha P_y \leq 0.4 P_y \)

\[
R_n = 0.60F_yd_fd_w (J11-9)
\]

2. For \( \alpha P_y > 0.4 P_y \)

\[
R_n = 0.60F_yd_fd_w \left(1 + \frac{\alpha P_y}{P_y}\right) (J11-10)
\]

(b) When the effect of inelastic panel-zone deformation on frame stability is accounted for in the analysis:

1. For \( \alpha P_y \leq 0.75 P_y \)

\[
R_n = 0.60F_yd_fd_w \left(1 + \frac{3b_ft_c^2}{d_fd_w}\right) (J11-11)
\]

2. For \( \alpha P_y > 0.75 P_y \)

\[
R_n = 0.60F_yd_fd_w \left(1 + \frac{3b_ft_c^2}{d_fd_w}\right) \left(1 + \frac{1.2\alpha P_y}{P_y}\right) (J11-12)
\]

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)
- \( P_y \) = 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)
- \( b_f \) = width of column flange, in. (mm)
- \( d_b \) = depth of beam, in. (mm)
- \( d_c \) = depth of column, in. (mm)
- \( t_f \) = 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)
- \( \alpha = 1.0 \) (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 or plate girder flange(s) shall be designed as axially compressed members (columns) in accordance with the requirements of Section E6.2 and Section J4.4. The member properties shall be determined using an effective length of \(0.75h\) and a cross section composed of two stiffeners, and a strip of the web having a width of \(0.94t_w\sqrt{E/F_y}\) 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, $\phi R_n$, and the allowable strength, $R_n/\Omega$, for the limit state of square and rectangular HSS under compressive concentrated forces shall be determined as:

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

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

$$R_n = C_{tw}^2 F_y \left(1 - C_r \frac{r}{t_w}\right) \left(1 + C_t \frac{l_b}{t_w}\right) \left(1 - C_b \frac{h}{t_w}\right)$$  \hspace{1cm} (J12-1)

where

- $C = \text{coefficient from Table J12.1}$
- $t_w = \text{design wall thickness, as defined in Section B4.2 of web, in. (mm)}$
- $F_y = \text{specified minimum yield stress, ksi (MPa)}$
- $C_r = \text{internal bend radius coefficient from Table J12.1}$
- $r = \text{internal radius of corner, in. (mm)}$
- $C_t = \text{bearing length coefficient from Table J12.1}$
- $l_b = \text{length of bearing, in. (mm)}$
- $C_b = \text{web slenderness coefficient from Table J12.1}$
- $h = \text{clear distance between flanges less the corner radius, in. (mm)}$
**TABLE J12.1**

Coefficients for Cold-Formed Square and Rectangular HSS

<table>
<thead>
<tr>
<th>Cases</th>
<th>C</th>
<th>Cr</th>
<th>Cl</th>
<th>Ch</th>
</tr>
</thead>
<tbody>
<tr>
<td>End One-Flange loading (EOF)</td>
<td>4</td>
<td>0.32</td>
<td>1.60</td>
<td>0.040</td>
</tr>
<tr>
<td>Interior One-Flange loading (IOF)</td>
<td>2</td>
<td>0.04</td>
<td>2.30</td>
<td>0.001</td>
</tr>
<tr>
<td>End Two-Flange loading (ETF)</td>
<td>2</td>
<td>0.35</td>
<td>2.60</td>
<td>0.050</td>
</tr>
<tr>
<td>Interior Two-Flange loading (ITF)</td>
<td>8</td>
<td>0.21</td>
<td>0.75</td>
<td>0.010</td>
</tr>
</tbody>
</table>

Table J12.1 shall apply to cold-formed square and rectangular HSS where:

- \( h/t_w \leq 60 \),
- \( b/t_w \leq 55 \),
- \( b/h \leq 3 \),
- \( r/t_w \leq 2.5 \)
TABLE J12.2
Loading Cases for Cold-Formed Square and Rectangular HSS

<table>
<thead>
<tr>
<th>Case</th>
<th>Illustrations</th>
</tr>
</thead>
<tbody>
<tr>
<td>EOF</td>
<td><img src="image1" alt="EOF Illustration" /> $l_{end} \leq 1.5h$</td>
</tr>
<tr>
<td>IOF</td>
<td><img src="image2" alt="IOF Illustration" /> $l_{end} &gt; 1.5h$</td>
</tr>
<tr>
<td>ETF</td>
<td><img src="image3" alt="ETF Illustration" /> $l_{end} \leq 1.5h$, $l_{spac} \leq 1.5h$</td>
</tr>
<tr>
<td>ITF</td>
<td><img src="image4" alt="ITF Illustration" /> $l_{end} &gt; 1.5h$, $l_{spac} \leq 1.5h$</td>
</tr>
</tbody>
</table>

**End One-Flange Loading (EOF):** distance from the bearing edge to the end of the member $\leq 1.5h$, 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 $\leq 1.5h$, and the clear distance between the bearing edges of adjacent opposite concentrated forces or reactions $\leq 1.5h$.

**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 $\leq 1.5h$. 

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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:

K2. HSS-to-HSS Truss Connections

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, $\Phi P_n$, and the allowable strength, $P_n/\Omega$, 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, \( P \sin \theta \), 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, \( P \sin \theta \), 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, \( P \sin \theta \), 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

\[ A_g = \text{gross area of member, in.}^2 (\text{mm}^2) \]
\[ B = \text{overall width of square HSS main member, measured 90° to the plane of the connection, in. (mm)} \]
\[ B_b = \text{overall width of square HSS branch member or plate, measured 90° to the plane of the connection, in. (mm)} \]
\[ B_e = \text{effective width of square HSS branch member or plate, in. (mm)} \]
\[ D = \text{outside diameter of round HSS main member, in. (mm)} \]
\[ D_b = \text{outside diameter of round HSS branch member, in. (mm)} \]
\[ F_c = \text{available stress in main member, ksi (MPa)} \]
\[ F = F_y \text{ for LRFD; 0.60}F_y \text{ for ASD} \]
\[ F_s = \text{specified minimum tensile strength of HSS member material, ksi (MPa)} \]
\[ F_y = \text{specified minimum yield stress of HSS main member material, ksi (MPa)} \]
\[ F_{yb} = \text{specified minimum yield stress of HSS branch member or plate material, ksi (MPa)} \]
\[ l_{od} = \text{distance from the near side of the connecting branch to end of chord, in. (mm)} \]

\[ t = \text{design wall thickness of HSS main member, as defined in Section B4.2, in. (mm)} \]

\[ t_b = \text{design wall thickness of HSS branch member, as defined in Section B4.2, in. (mm)} \]

\[ O_v = \frac{l_{ov}}{l_p} \times 100, \% \]

\[ e = \text{eccentricity in a truss connection, positive being away from the branches, in. (mm)} \]

\[ g = \text{gap between toes of branch members in a gapped K-connection, neglecting the welds, in. (mm)} \]

\[ l_{ov} = \text{overlap length measured along the connecting face of the chord beneath the two branches, in. (mm)} \]

\[ l_p = \text{projected length of the overlapping branch on the chord, in. (mm)} \]

\[ \beta = \text{width ratio; the ratio of branch diameter to chord diameter} = \frac{D_b}{D} \text{ for round HSS; the ratio of overall branch width to chord width} = \frac{B_b}{B} \text{ for square HSS} \]

\[ \beta_{eff} = \text{effective width ratio; the sum of the perimeters of the two branch members in a K-connection divided by eight times the chord width} \]

\[ \gamma = \text{chord slenderness ratio; the ratio of one-half the diameter to the thickness} = \frac{D}{2t} \text{ for round HSS; the ratio of one-half the width to thickness} = \frac{B}{2t} \text{ for square HSS} \]

\[ \theta = \text{acute angle between the branch and chord (degrees)} \]

\[ \zeta = \text{gap ratio; the ratio of the gap between the branches of a gapped K-connection to the width of the chord} = \frac{g}{B} \text{ for square HSS} \]

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 state associated with the strength equations given in Table K2.1. Therefore, if deformations at service loads are critical for this type 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 shall be made. This analysis shall take account of the secondary moments in the joints caused by the bending stiffness of the joints.

3. **Square HSS**

The available strength, \( \phi P_n \) and \( P_n / \Omega \), 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.
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:

$$B_e = \frac{10}{B} \left( \frac{F_{t,t}}{F_{y,t,b}} \right) B_b \leq B_b \quad (K2-1)$$
### TABLE K2.1
Available Strengths of Round HSS-to-HSS Truss Connections

<table>
<thead>
<tr>
<th>Connection Type</th>
<th>Connection Available Axial Strength</th>
</tr>
</thead>
</table>
| General Check for T-, Y-, Cross- and K-Connections with gap, when $D_{h\text{ (tens/comp)}} < (D - 2t)$ | Limit state: shear yielding (punching)  
\[ P_{n} = 0.6 F_{y} t_{n} D_{h} \left( \frac{1 + \sin \theta}{2 \sin^{2} \theta} \right) \] (K2-2)  
\[ \phi = 0.95 \text{ (LRFD)} \quad \Omega = 1.58 \text{ (ASD)} \] |
| T- and Y-Connections                    | Limit state: chord plastification  
\[ P_{n} \sin \theta = F_{y} t_{n}^{2} \left( 3.1 + 15.6 \beta^{2} \right) t_{n}^{0.2} Q_{f} \] (K2-3)  
\[ \phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)} \] |
| Cross-Connections                      | Limit state: chord plastification  
\[ P_{n} \sin \theta = F_{y} t_{n}^{2} \left( \frac{5.7}{1 - 0.81 \beta} \right) Q_{f} \] (K2-4)  
\[ \phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)} \] |
| K-Connections with gap or overlap      | Limit state: chord plastification  
\[ (P_{n} \sin \theta)_{\text{compression branch}} = F_{y} t_{n}^{2} \left( 2.0 + 11.33 \frac{D_{h\text{ (tens/comp)}}}{D} \right) Q_{f} \] (K2-5)  
\[ (P_{n} \sin \theta)_{\text{tension branch}} = (P_{n} \sin \theta)_{\text{compression branch}} \] (K2-6)  
\[ \phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)} \] |
 FUNCTIONS

\[ Q_s = 1 \] for chord (connecting surface) in tension

\[ Q_s = 1.0 - 0.3U \left( 1 + U \right) \] for HSS (connecting surface) in compression \hspace{1cm} (K2-7)

\[ U = \frac{P_o}{F_{A_s}} + \frac{M_o}{F_s} \] \hspace{1cm} (K2-8)

where \( P_o \) and \( M_o \) are determined on the side of the joint that has the lower compression stress.

\( P_o \) and \( M_o \) refer to required strengths in the HSS: \( P_o = P_t \) for LRFD, and \( P_o \) for ASD; \( M_o = M_u \) for LRFD, and \( M_o \) for ASD.

\[ Q_s = r^2 \left[ 1 + \frac{0.024r^2}{\exp\left(\frac{0.5r^2}{T} - 1.33\right) + 1} \right] \] \hspace{1cm} (K2-9)

Note that \( \exp(x) \) is equal to \( e^x \), where \( e=2.71828 \) is the base of the natural logarithm.

<table>
<thead>
<tr>
<th>TABLE K2.1A</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Limits of Applicability of Table K2.1</strong></td>
</tr>
</tbody>
</table>

| Joint eccentricity: | \(-0.55 \leq \varepsilon \leq 0.25\) for K-connections |
| Chord wall slenderness: | \( D/t \) ≤ 50 for T-, Y- and K-connections |
| Branch wall slenderness: | \( D/t \) ≤ 40 for cross-connections |
| Width ratio: | \( D_b/f_b \) ≤ 50 for tension and compression branch |
| Gap: | \( D_b/f_b \) ≤ 0.05E/F_{thb} for compression branch |
| Overlap: | \( g \) ≥ \( b_{comp} + b_{tens} \) for gapped K-connections |
| Branch thickness: | \( -25\% \) ≤ \( O_t \leq 100\% \) for overlapped K-connections |
| End distance: | \( t_{overlapping} \) ≤ \( t_{overlapped} \) for branches in overlapped K-connections |
| | \( l_{end} \) ≥ \( D \left( 1.25 \frac{b}{2} \right) \) for T-, Y-, cross- and K-connections |
### TABLE K2.2
Available Strengths of Square HSS-to-HSS Truss Connections

<table>
<thead>
<tr>
<th>Connection Type</th>
<th>Connection Available Axial Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Gapped K-Connections</strong></td>
<td>Limit state: chord wall plastification, for all $\beta$</td>
</tr>
<tr>
<td></td>
<td>$P_b \sin \theta = F_f t^2 \left(9.8\beta_{eff} \gamma^{0.5}\right) \phi_f$</td>
</tr>
<tr>
<td></td>
<td>$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)</td>
</tr>
</tbody>
</table>

**Overlapped K-Connections**

Limit state: Local Yielding of Branch/Branches due to Uneven Load Distribution

$\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)

When $25\% \leq O_e < 50\%$

$P_{e,i} = F_{j,sh/tb} \left[\frac{O_e}{50}(2B_{sh} - 4t_{sh}) + B_{ei} + B_{ij}\right]$  \hspace{1cm} (K2-11)

When $50\% \leq O_e < 80\%$

$P_{e,j} = F_{j,sh/tb} \left(2B_{sh} - 4t_{sh} + B_{ei} + B_{ij}\right)$  \hspace{1cm} (K2-12)

When $80\% \leq O_e \leq 100\%$

$P_{e,j} = F_{j,sh/tb} \left(2B_{sh} - 4t_{sh} + B_{ei} + B_{ij}\right)$  \hspace{1cm} (K2-13)

Subscript $i$ refers to the overlapping branch
Subscript $j$ refers to the overlapped branch

$P_{e,j} = P_{e,i} \left(\frac{F_{j,sh/tb}}{F_{i,sh/tb}}\right)$  \hspace{1cm} (K2-14)

**Functions**

$$Q_f = 1$$ for chord (connecting surface) in tension

$$= 1.3 - 0.4 \frac{U}{\beta_{eff}} \leq 1.0$$ for chord (connecting surface) in compression, for gapped K-connections  \hspace{1cm} (K2-15)

$$U = \frac{P_{ro}}{F_c A_g} + \frac{M_{ro}}{F_c S}$$  \hspace{1cm} (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:

$P_{ro} = P_b$ for LRFD, and $P_b$ for ASD; $M_{ro} = M_b$ for LRFD, and $M_b$ for ASD.

$$\beta_{eff} = \left[\left(B_b\right)_{compression\ branch} + \left(B_b\right)_{tension\ branch}\right] / 2B$$  \hspace{1cm} (K2-16)

$$\beta_{exp} = \frac{\frac{\beta}{\gamma}}{\beta}$$  \hspace{1cm} (K2-17)
| **TABLE K2.2A**  
| Limits of Applicability of Table K2.2 |  
| **Joint eccentricity:** | $-0.55 \leq \frac{e}{B} \leq 0.25$ for K-connections |  
| **Chord wall slenderness:** | $B/t \leq 35$ for gapped K-connections |  
| **Branch wall slenderness:** | $B_e/t_e \leq 35$ for tension branch |  
| | $\leq 1.25 \frac{E}{F_{pe}}$ for compression branch of gapped K-connections |  
| | $\leq 35$ for compression branch of gapped K-connections |  
| | $\leq 1.1 \frac{E}{F_{pe}}$ for compression branch of overlapped K-connections |  
| **Width ratio:** | $B_x/B \geq 0.25$ for overlapped K-connections |  
| **Overlap:** | $25\% \leq \frac{O_i}{100\%}$ for overlapped K-connections |  
| **Branch width ratio:** | $B_{ik}/B_{ij} \geq 0.75$ for overlapped K-connections, where subscript $i$ refers to the overlapping branch and subscript $j$ refers to the overlapped branch |  
| **Branch thickness ratio:** | $t_{oi}/t_{oj} \leq 1.0$ 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:** | $\frac{B_o}{B} \geq 0.1 + \frac{\gamma}{50}$ |  
| **Gap ratio:** | $\zeta = g/B \geq 0.5(1-\beta_{eff})$ |  
| | $\zeta = g/B \leq 1.5(1-\beta_{eff})$ |  
| **Gap:** | $g \geq t_o\text{ compression branch} + t_o\text{ tension branch}$ |  
| **Branch size:** | smaller $B_o \geq 0.63(\text{larger } B_b)$ |  

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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:

L2. Tension Members
L2.1. Deflections
L2.5. Drift
L2.6. Vibration
L2.9. Wind-Induced Motion
L2.10. Thermal Expansion and Contraction
L2.11. Connection Slip

User Note: Stainless steel is usually specified because of its corrosion resistance or for aesthetic reasons. Section A3.1c 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 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 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. TENSION MEMBERS

For structures that are sensitive to deformations at serviceability, the tensile strength, $P_{n}$, of austenitic and duplex stainless steel tension members is permitted to be determined under service loads as:

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where
\[ A_r = \text{effective net area, in.}^2 (\text{mm}^2) \]
\[ F_y = \text{specified minimum yield stress, ksi (MPa)} \]

### L23. 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 L23-1.

\[ E_r = \frac{E_{st} + E_{sc}}{2} \]  \hspace{1cm} (L23-1)

where

\[ E_{st} = \text{secant modulus corresponding to the maximum compressive stress in the cross section, which may be determined in accordance with Appendix A Equation A-7-4, ksi (MPa)} \]

\[ E_{sc} = \text{secant modulus corresponding to the maximum tensile stress in the cross section, which may be determined in accordance with Appendix A Equation A-7-4, ksi (MPa)} \]

**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 L23.1 gives the secant modulus for common types of stainless steel at a maximum stress in the cross section equal to 0.6 \( F_y \), which may be conservatively adopted in preliminary estimates of deflection.

<table>
<thead>
<tr>
<th>Stainless Steel</th>
<th>( F_y ) ksi (MPa)</th>
<th>( E_{sy} ) ksi (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Austenitic</td>
<td>S30400 and S31600</td>
<td>30 (205)</td>
</tr>
<tr>
<td></td>
<td>S30403 and S31603</td>
<td>25 (170)</td>
</tr>
<tr>
<td>Duplex</td>
<td>S32101, S32202 and S32205</td>
<td>65 (450)</td>
</tr>
<tr>
<td></td>
<td>S32304</td>
<td>58 (400)</td>
</tr>
</tbody>
</table>

### L34. DRIFT

Drift shall be limited so as not to impair the serviceability of the structure.

### L45. 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.

**WIND-INDUCED MOTION**

The effect of wind-induced motion of buildings on the comfort of occupants shall be considered.

**THERMAL EXPANSION AND CONTRACTION**

The effects of thermal expansion and contraction of a building shall be considered.

**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 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 carbon 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 surface contamination by carbon steel and iron or other materials contamination due to exposure to steel cutting or grinding, and other potential sources of damage that may adversely affect appearance or performance. Suitable measures include but are not limited to:

- 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 shall 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 shall be removed as soon as possible to prevent
crevice corrosion under the film. Only UV rated film shall be exposed to sunlight.

- Avoidance of unprotected storage or transportation in locations exposed to salt-laden coastal atmospheres or deicing salt mist. Precautions shall also be taken to avoid immersion in seawater, as can occur during a storm surge.

- Protection of storage racks by wooden, rubber or plastic battens or sheaths to avoid free iron contamination, copper, zinc or other metallic contact with stainless steel surfaces.

- Protection of stainless steel from direct contact with carbon steel lifting tackle 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 shall be used to ensure that surface contamination does not occur.

- 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 shall be determined.

- Use of segregated manufacturing areas for stainless steel to prevent free iron contamination or other metals from being embedded in the surface.

- 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 shall be stainless steel, preferably an austenitic stainless steel of equivalent corrosion resistance.

**User Note:** 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 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.

**User Note:** See AISC Design Guide 27 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 general requirements standards listed in Section A3 for the product or as agreed in the contract documents.
User Note: 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 carbon steel 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 maybe 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 duplexes and austenitic stainless steels alloyed with nitrogen, require higher forming forces and show more elastic springback.

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, 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 1/8 in. (3 mm) of material is mechanically removed from any thermally cut edge. Oxy-acetylene torch cutting shall not be used.

Notches or gouges on cut surfaces (edges) not exceeding 1/16 in. (2 mm) for materials less than 5/8 in. (16 mm) in thickness or 10% of the materials thickness for materials 5/8 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 engineer 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 1/8 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 EORengineer. 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 Part C Acceptance Criteria shall be used to evaluate discontinui-
ties 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 un-
less specifically called for in the construction documents or included in a stip-
ulated 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 which 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 an-
other suitable method in accordance with ASTM A380/A380M. The contract
documents The engineer shall specify a cleaning procedure in accordance with
ASTM A380/A380M that is suitable for the corrosiveness of the service envi-
ronment. Chemical passivation and electropolishing are not permissible meth-
ods of restoring weld corrosion resistance unless combined with methods suit-
able for removing the chromium depleted layer.

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 contam-
nation.

User Note: Duplex stainless steel requires careful control and monitoring of the
heat input during welding and may require postweld heat treatment annealing
per the material specification. The purpose of this heat treatment is to match the
corrosion resistance of the base metal. Use of an appropriate overalloyed filler
metal in accordance with AWS D1.6/D1.6M or the alloy producer’s recommen-
dation if no guidance is given by AWS D1.6/D1.6M may make postweld heat
treatment unnecessary but corrosion testing should be considered for verifica-
tion unless the product or process has been prequalified.

User Note: Weld distortion is generally greater in stainless steel than in other
carbon 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 an appropriate weld procedure approved by the EORengineer.
Bimetallic interfaces in welded joints combining stainless steels with other steel carbon or alloy steel 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 carbon 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 1/8 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 engineer shall establish limits for notches and gouges in areas with stress concentrations.

Fully inserted finger shims, with a total thickness of not more than 1/4 in. (6 mm) within a joint, are permitted without changing the strength (based upon hole type) for the design of connections. The orientation of such shims is independent of the direction of application of the load.

The use of bolts shall conform to the requirements of Section J3.

In bolted joints between carbon steel and stainless steel in potentially corrosive environments, provision shall be made to electrically isolate the carbon steel and stainless steel elements. This entails the use of insulating washers, sealants around the joints to prevent moisture from connecting the metals or possibly bushings. The insulating washers and bushings shall be made of a thermoset polymer such as neoprene (synthetic rubber), which is flexible enough to seal the joint when adequate pressure is applied and long lasting to provide permanent metal separation.

**User Note:** When insulating washers and bushing are used to provide corrosion protection in snug-tightened connections, the product used in the bolt grip will need to be reviewed and approved by the engineer. The amount of tightening required in the connection will need to be provided by the manufacturer.

In addition to joints between carbon and stainless steel, if there is a connection to another metal(s) then the engineer shall consider the possible galvanic effects associated with the metal combination and the longevity of any isolating barrier if there are significant differences in coefficient of thermal expansion.
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 Chapter 6 of the Code of Standard Practice for Structural Stainless Steel Buildings. Dimensional tolerance for structural shapes shall be specified in accordance with Section A3.1b(g) and (h).

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 $R_z$ or $R_s$. 

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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 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:

B1. The representative sample shall have a minimum thickness of at least 1/4 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: Class SSA surfaces may not require qualification of a blast process since as-rolled steel can generally meet these surface roughness requirements.

Clean stainless steel shot and grit media shall be used when blast cleaning these surfaces to avoid contamination.

<table>
<thead>
<tr>
<th>TABLE M2.1 Definition of Surface Classes for Slip Critical Faying Surfaces</th>
</tr>
</thead>
<tbody>
<tr>
<td>SSA[^2]</td>
</tr>
<tr>
<td>SSB</td>
</tr>
<tr>
<td>SSC</td>
</tr>
<tr>
<td>SSD</td>
</tr>
</tbody>
</table>

[^2]: Rz is the surface roughness according to ASTM D7127.
[^3]: Rt is the surface roughness according to ASTM D4417.

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 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 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 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 (PJP groove welded or bolted), is permitted. If the gap exceeds 1/16 in. (2 mm), but is equal to or less than 1/4 in. (6 mm), and if an engineering investigation shows that sufficient contact area does not exist, the gap shall be packed out with nontapered steel shims. Shims 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 carbon steel shall be avoided. Other carbon steel alloy brushes, steel wool, steel scrapers or other carbon steel 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:

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 specifications, standards, if applicable, and Section 6.1 of the Code of Standard Practice for Structural Stainless Steel Buildings, 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 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 Code of Standard Practice for Structural Stainless Steel Buildings, Section 6.4. Dimensional tolerance for structural shapes shall be specified in accordance with Section A3.1b (g) and (h).

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 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 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 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

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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 specification (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, QC inspection, testing, or witness qualification(s).

(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 clause 7 of AWS B5.1 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 carbon steel alloy and stainless steel fasteners including the issues related to bimetallic contact between dissimilar metals.

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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 both other carbon steel alloys and stainless steel fasteners 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 Code of Standard Practice for Structural Stainless Steel Buildings requires the transfer of information from the contract documents.
2. Quality Assurance

The QAI shall review the material test reports and certifications as listed in Section N3.2 for compliance with the construction documents.

QA inspection tasks shall be performed by the QAI, in accordance with Sections N5.4, N5.6 and N5.7.

Tasks in Tables N5.4-1 through N5.4-3 and N5.6-1 through N5.6-3 listed for QA are those inspections performed by the QAI to ensure that the work is performed in accordance with the construction documents.

Concurrent with the submittal of such reports to the AHJ, EOR or owner, the QA agency shall submit to the fabricator and erector:

(a) Inspection reports
(b) NDT reports

3. Coordinated Inspection

When a task is noted to be performed by both QC and QA, it is permitted to coordinate the inspection function between the QCI and QAI so that the inspection functions are performed by only one party. When QA relies upon inspection functions performed by QC, the approval of the EOR and the AHJ is required.

4. Inspection of Welding

Observation of welding operations and visual inspection of in-process and completed welds shall be the primary method to confirm that the materials, procedures and workmanship are in conformance with the construction documents.

**User Note:** The technique, workmanship, appearance and quality of welded construction are addressed in Section M2.4 and 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.
## Table N5.4-1
**Inspection Tasks Prior to Welding**

<table>
<thead>
<tr>
<th>Inspection Tasks Prior to Welding</th>
<th>QC</th>
<th>QA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Welder qualification records and continuity records</td>
<td>P</td>
<td>O</td>
</tr>
<tr>
<td>WPS available</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td>Manufacturer certifications for welding consumables available</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td>Material identification (type/grade)</td>
<td>O</td>
<td>O</td>
</tr>
<tr>
<td>Welder identification system[^a]</td>
<td>O</td>
<td>O</td>
</tr>
<tr>
<td>Verify handling procedures to avoid contamination</td>
<td>O</td>
<td>O</td>
</tr>
<tr>
<td>Fit-up of groove welds (including joint geometry)</td>
<td>O</td>
<td>O</td>
</tr>
<tr>
<td>- Joint preparations</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Dimensions (alignment, joint root opening, joint root face, bevel)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Cleanliness (condition of steel surfaces)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Tacking (tack weld quality and location)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Backing type and fit (if applicable)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fit-up of CJP groove welds of HSS K-joints without backing (including joint geometry)</td>
<td>P</td>
<td>O</td>
</tr>
<tr>
<td>- Joint preparations</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Dimensions (alignment, joint root opening, joint root face, bevel)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Cleanliness (condition of steel surfaces)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Tacking (tack weld quality and location)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Configuration and finish of access holes</td>
<td>O</td>
<td>O</td>
</tr>
<tr>
<td>Fit-up of fillet welds</td>
<td>O</td>
<td>O</td>
</tr>
<tr>
<td>- Dimensions (alignment, gaps at joint root)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Cleanliness (condition of steel surfaces)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Tacking (tack weld quality and location)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Check welding equipment</td>
<td>O</td>
<td>-</td>
</tr>
</tbody>
</table>

[^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 EORengineer and shall be the low-stress type and compatible with the stainless steel base metal.
### TABLE N5.4-2
Inspection Tasks During Welding

<table>
<thead>
<tr>
<th>Inspection Tasks During Welding</th>
<th>QC</th>
<th>QA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control and handling of welding consumables</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Packaging</td>
<td>O</td>
<td>O</td>
</tr>
<tr>
<td>• Exposure control</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No welding over cracked tack welds</td>
<td>O</td>
<td>O</td>
</tr>
<tr>
<td>Environmental conditions</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Wind speed within limits</td>
<td>O</td>
<td>O</td>
</tr>
<tr>
<td>• Precipitation and temperature</td>
<td></td>
<td></td>
</tr>
<tr>
<td>WPS followed</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Settings on welding equipment</td>
<td>O</td>
<td>O</td>
</tr>
<tr>
<td>• Travel speed</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Selected welding materials</td>
<td>O</td>
<td>O</td>
</tr>
<tr>
<td>• Shielding gas type/flow rate</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Preheat applied</td>
<td>O</td>
<td>O</td>
</tr>
<tr>
<td>• Interpass temperature maintained (min./max.)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Proper position (F, V, H, OH)</td>
<td>O</td>
<td>O</td>
</tr>
<tr>
<td>Welding techniques</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Interpass and final cleaning</td>
<td>O</td>
<td>O</td>
</tr>
<tr>
<td>• Each pass within profile limitations</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Each pass meets quality requirements</td>
<td>O</td>
<td>O</td>
</tr>
</tbody>
</table>

### TABLE N5.4-3
Inspection Tasks After Welding

<table>
<thead>
<tr>
<th>Inspection Tasks After Welding</th>
<th>QC</th>
<th>QA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Welds cleaned</td>
<td>O</td>
<td>O</td>
</tr>
<tr>
<td>Size, length and location of welds</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td>Welds meet visual acceptance criteria</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Crack prohibition</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Weld/base-metal fusion</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Crater cross section</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Weld profiles</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td>Weld size</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Undercut</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Porosity</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Arc strikes</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td>k-area(^a)</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td>Weld access holes in built-up heavy shapes(^b)</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td>Backing removed and weld tabs removed (if required)</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td>Repair activities</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td>Document acceptance or rejection of welded joint or member</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td>No prohibited welds have been added without the approval of the EOR</td>
<td>O</td>
<td>O</td>
</tr>
</tbody>
</table>

\(^a\) 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 A3.1d) are welded, visually inspect the weld access hole for cracks.
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 M2.4 and M2.6.

**User Note:** MT is not an acceptable inspection method for austenitic stainless steels due to their non-magnetic 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.

5b. CJP Groove Weld NDT

For structures in risk category III or IV, UT shall be performed by QA on all complete-joint-penetration (CJP) groove welds subject to transversely applied tension loading in butt, T- and corner joints, in material 5/16 in. (8 mm) thick or greater. For structures in risk category II, UT shall be performed by QA on 10% of CJP groove welds in butt, T- and corner joints subject to transversely applied tension loading, in materials 5/16 in. (8 mm) thick or greater.

**User Note:** For structures in risk category I, NDT of CJP groove welds is not required. For all structures in all risk categories, NDT of CJP groove welds in materials less than 5/16 in. (8 mm) thick is not required.

5c. Welded Joints Subjected to Fatigue

When required by Appendix 3, Table A-3.1, welded joints requiring weld soundness to be established by radiographic or ultrasonic inspection shall be tested by QA as prescribed. Reduction in the rate of UT is prohibited.

5d. Ultrasonic Testing Rejection Rate

The ultrasonic testing rejection rate shall be determined as the number of welds containing defects divided by the number of welds completed.
that contain acceptable discontinuities shall not be considered as having de-
fects 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 consid-
ered 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 ap-
proved 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 demon-
strated to be 5% or less of the welds tested for the welder or welding opera-
tor. A sampling of at least 40 completed welds shall be made for such re-
duced evaluation on each project.

5f. Increase in Ultrasonic Testing Rate

For structures in risk category II and higher (where the initial rate for UT is
10%) the NDT rate for an individual welder or welding operator shall be
increased to 100% should the rejection rate (the number of welds containing
unacceptable defects divided by the number of welds completed) exceed 5%
of the welds tested for the welder or welding operator. A sampling of at least
20 completed welds on each project shall be made prior to implementing
such an increase. If the rejection rate for the welder or welding operator falls
to 5% or less on the basis of at least 40 completed welds, the rate of UT may
be decreased to 10%.

5g. Documentation

All NDT performed shall be documented. For shop fabrication, the NDT
report shall identify the tested weld by piece mark and location in the piece.
For field work, the NDT report shall identify the tested weld by location in
the structure, piece mark, 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 con-
firm that the materials, procedures and workmanship incorporated in con-
struction are in conformance with the construction documents and the appli-
cable 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 spec-
ified 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.

(cb) 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.

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>
<thead>
<tr>
<th>Inspection Tasks Prior to Bolting</th>
<th>QC</th>
<th>QA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Manufacturer’s certifications available for fastener materials</td>
<td>O</td>
<td>P</td>
</tr>
<tr>
<td>Fasteners marked in accordance with ASTM requirements</td>
<td>O</td>
<td>O</td>
</tr>
<tr>
<td>Correct fasteners selected for the joint detail (grade, type, bolt length if threads are to be excluded from shear plane)</td>
<td>O</td>
<td>O</td>
</tr>
<tr>
<td>Correct bolting procedure selected for joint detail</td>
<td>O</td>
<td>O</td>
</tr>
<tr>
<td>Connecting elements, including the appropriate faying surface condition and hole preparation, if specified, meet applicable requirements</td>
<td>O</td>
<td>O</td>
</tr>
<tr>
<td>Pre-installation verification testing by installation personnel observed and documented for fastener assemblies and methods used</td>
<td>P</td>
<td>O</td>
</tr>
<tr>
<td>Protected storage provided for bolts, nuts, washers and other fastener components</td>
<td>O</td>
<td>O</td>
</tr>
</tbody>
</table>
TABLE N5.6-2
Inspection Tasks During Bolting

<table>
<thead>
<tr>
<th>Inspection Tasks During Bolting</th>
<th>QC</th>
<th>QA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fastener assemblies placed in all holes and washers and nuts are positioned as required</td>
<td>O</td>
<td>O</td>
</tr>
<tr>
<td>Joint brought to the snug-tight condition prior to the pretensioning operation</td>
<td>O</td>
<td>O</td>
</tr>
<tr>
<td>Fastener component not turned by the wrench prevented from rotating</td>
<td>O</td>
<td>O</td>
</tr>
<tr>
<td>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</td>
<td>O</td>
<td>O</td>
</tr>
</tbody>
</table>

TABLE N5.6-3
Inspection Tasks After Bolting

<table>
<thead>
<tr>
<th>Inspection Tasks After Bolting</th>
<th>QC</th>
<th>QA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Document acceptance or rejection of bolted connections</td>
<td>P</td>
<td>P</td>
</tr>
</tbody>
</table>

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 carbon steel alloys or stainless steel) and other embedments supporting other structural carbon 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

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

Specification for Structural Stainless Steel Buildings, xxx
Public Review draft dated October 14, 2020
AMERICAN INSTITUTE OF STEEL CONSTRUCTION
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 \( P-\Delta \), \( P-\delta \) 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 \( P-\Delta \) and \( P-\delta \) 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 \( L_f \). The type of second-order analysis defined herein also includes the beneficial effects of additional member torsional strength and stiffness due to warping restraint, which can be conservatively neglected. Refer to the Commentary for additional information and guidance.

(c) In all cases, the analysis shall directly model the effects of initial imperfections due to both points of intersection of members displaced from their nominal locations (system imperfections), and initial out-of-straightness or offsets of members along their length (member imperfections). The magnitude of the initial displacements shall be the maximum amount considered in the design; the pattern of initial displacements shall be such that it provides the greatest destabilizing effect for the load combination being considered. The use of notional loads to represent either type of imperfection is not permitted.

User Note: Initial displacements similar in configuration to both displacements due to loading and anticipated buckling modes should be considered in the modeling of imperfections. The magnitude of the initial points of intersection of members displaced from their nominal locations (system imperfections) should be based on permissible construction tolerances, as specified in the 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 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. Refer to the Commentary for additional guidance.

2b. Adjustments to Stiffness

The analysis of the structure to determine the required strengths of components shall use reduced stiffnesses as defined in Section C2.3. Such stiffness reduction, including the general stiffness reduction factors, \( \tau_g \) and \( \tau_b \), shall be applied to all stiffnesses that are considered to contribute to the stability of the structure. The use of notional loads to represent \( \tau_g \) and \( \tau_b \) is not permitted.

User Note: Stiffness reduction should be applied to all member properties including torsional properties \((GJ, EC_w)\) 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.

Specification for Structural Stainless Steel Buildings, xxx
Public Review draft dated October 14, 2020
American Institute of Steel Construction
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, \( F_y A_g \), or as \( F_y A_e \) 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

The nominal compressive strength of members, \( P_n \), may be taken as the cross section compressive strength, \( F_y A_g \), or as \( F_y A_e \) 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. 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 provisions of Section 1.3 do not apply to seismic design.

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 \( P-\Delta \), \( P-\delta \) 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.

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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, in-
cluding I-shapes, HSS and box sections.

2. Ductility Requirements

Members and connections with elements subject to yielding shall be propor-
tioned 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 defor-
mation capacities, the following requirements shall be satisfied for steel mem-
bers subject to inelastic deformation.

2a. Material

The specified minimum yield stress, \( F_y \), of members subject to inelastic defor-
mation shall not exceed 80 ksi (550 MPa).

2b. Unbraced Length

In prismatic member segments that experience significant plasticity \( (M > M_p) \), 
the laterally unbraced length, \( L_b \), shall not exceed \( L_{p_0d} \) 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 sec-
tion. 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:

\[
L_{p_0d} = \left( 1.83 - 1.17 \frac{M_1'}{M_2} \right) L_p
\]

where

\[
L_p = \text{limiting laterally unbraced length for the limit state of yielding},
\]

as defined by Equation F2-7, in. (mm),

(1) When the magnitude of the bending moment at any location within 
the unbraced length exceeds \( M_2 \)

\[
M_1'/M_2 = +1
\]

(A-1-6a)

Otherwise:

(2) When \( M_{mid} \leq (M_1 + M_2)/2 \)

\[
M_1' = M_1
\]

(A-1-6b)

(3) When \( M_{mid} > (M_1 + M_2)/2 \)
\[ M_1' = (2M_{mid} - M_2) < M_2 \]  
(A-1-6c)

where

- \( M_1 \) = smaller moment at end of unbraced length, kip-in. (N-mm)
- \( M_2 \) = larger moment at end of unbraced length, kip-in. (N-mm)
- \( M_{mid} \) = moment at middle of unbraced length, kip-in. (N-mm)

(shall be taken as positive in all cases)

\[ M_1' = \text{effective moment at end of unbraced length opposite from } \]

\( M_2 \), kip-in. (N-mm)

The moments \( M_1 \) and \( M_{mid} \) are individually taken as positive when they cause compression in the same flange as the moment, \( M_2 \), and taken as negative otherwise.

(b) For solid rectangular bars bent about their major axis

\[ L_{pd} = \left( 0.57 - 0.36 \frac{M_1'}{M_2} \right) \frac{E t^2}{F_y d} \]  
(A-1-7)

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 \( 4.71 \frac{E F_y}{y d} \) and \( 4.71 \frac{E F_y}{y d} \), 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 \( 0.75 F_y A_e \).

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 \( \varepsilon \) is less than or equal to the CSM strain limit \( \varepsilon_{csm} \) at that location, as discussed in Section 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 modelled 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 should be met:

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Where \( \varepsilon_r \) is the required compressive strain, averaged over the local buckling half-wavelength, \( L_e \), and \( \varepsilon_y = F_y/E \) is the yield strain, and the ratio \( \varepsilon_{cs} / \varepsilon_y \) shall be determined using Equations A-1-15 and A-1-16.

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 \( \lambda_i \leq 0.68 \)

\[
\frac{\varepsilon_{cs}}{\varepsilon_y} = 0.25 + \frac{0.002}{\lambda_i^{1.6}} \leq \Lambda
\]

(A-1-15)

When \( 0.68 < \lambda_i < 1.00 \)

\[
\frac{\varepsilon_{cs}}{\varepsilon_y} = \left(1 - \frac{0.222}{\lambda_i^{1.05}}\right) \frac{1}{\lambda_i^{1.05}} + \frac{0.002 (F/F_y)^n}{\varepsilon_y}
\]

(A-1-16)

where

\( f = \) stress at outer compressive fiber, ksi (MPa)
\( F_y = \) specified minimum yield stress, ksi (MPa)
\( \varepsilon_y = F_y/E \)
\( \varepsilon_{cs} = \) CSM strain limit
\( E = \) modulus of elasticity of stainless steel
\( \lambda_i = \) local cross-section slenderness
\( \Lambda = \) upper bound strain limit with a recommended value of 15

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 and Bending Interaction

The interaction between bending and shear shall be accounted for through a reduction factor, \( \rho_{cs} \), applied to the CSM strain limit, \( \varepsilon_{cs} \), as given by Equation A-1-18.

\[
\frac{\varepsilon_r}{\rho_{cs} \varepsilon_{cs}} \leq 1.0
\]

(A-1-18)

where \( \rho_{cs} \) shall be determined as follows:

When \( V_r \leq 0.5 V_c \)

\[ \rho_{cs} = 1.0 \]
When $V_r > 0.5 V_c$

\[ \rho_{\text{com}} = \frac{0.5}{\frac{1}{0.5} + \rho} \quad (A-1-20) \]

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, $\phi V_n$ or $V_n/\Omega$, determined in accordance with Chapter G, kips (N)

\[ \rho = \left( \frac{2V_r}{V_c} - 1 \right)^2 \quad (A-1-21) \]
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-shaped, channels, angles, tees, HSS, and box sections which 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.

TABLE A-2.1.1

<table>
<thead>
<tr>
<th>Range of applicability of the CSM</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Compression members</strong></td>
</tr>
<tr>
<td>I-shaped, channels, angles, tees, square and rectangular HSS, and box sections</td>
</tr>
<tr>
<td>$L &lt; 0.63 \frac{E}{r}$ or $F_L \leq 0.04 \times F_y$ and $\lambda_I \leq 1.6$</td>
</tr>
<tr>
<td>Round HSS</td>
</tr>
<tr>
<td>$L &lt; 0.63 \frac{E}{r}$ or $F_L \leq 0.04 \times F_y$ and $\lambda_I \leq 0.6$</td>
</tr>
</tbody>
</table>

(a) Limitations on member length shall not apply when designing according to Appendix 1.

where

$\lambda_I$ = cross-section slenderness, as defined in Section 2.3.2

$E$ = modulus of elasticity of stainless steel

28,000 ksi (193 000 MPa) for austenitic, and 29,000 ksi (200 000 MPa) for duplex stainless steel
2.2. MATERIAL MODELING

The elastic, linear hardening material model used with the CSM is shown in Figure A-2.2.1.

![Fig. A-2.2.1. CSM material model.]

where

- \( \varepsilon_y \) = yield strain determined according to Equation A-2-1
- \( \varepsilon_u \) = strain at the ultimate tensile stress determined according to Equation A-2-3
- \( F\varepsilon_y \) = strain hardening modulus determined according to Equation A-2-2

\[
F_{\varepsilon} = \frac{F_y}{E} \quad (A-2-1)
\]

\[
F\varepsilon = \frac{F_u - F_y}{0.16\varepsilon_u - \varepsilon_y} \quad (A-2-2)
\]

\[
\varepsilon_u = 1 - \frac{F_y}{F_u} \quad (A-2-3)
\]

2.3. DEFORMATION CAPACITY

The normalized deformation capacity, \( \varepsilon_{csb}/\varepsilon_y \), shall be determined as follows:

1. I-Shaped, Channels, Angles, Tees, Square and Rectangular HSS, and Box Sections

   (a) When \( \lambda_1 \leq 0.68 \)
2. Round HSS

(a) When \( \lambda_i \leq 0.30 \)

\[
\frac{e_{csm}}{e_y} = \frac{4.44 \times 10^{-3}}{\lambda_i^{4.5}} \leq \text{minimum} \left( \frac{\Lambda}{0.10e_u}, \frac{0.10e_u}{e_y} \right) \quad \text{(A-2-56)}
\]

(b) When \( \lambda_i > 0.30 \)

\[
\frac{e_{csm}}{e_y} = \left( 1 - \frac{0.224}{\lambda_i^{0.342}} \right) \frac{1}{\lambda_i^{0.342}} \quad \text{(A-2-67)}
\]

where

\[ e_{csm} = \text{cross section failure strain} \]

\[ \lambda_i = \text{cross section slenderness, determined according to Equation A-2-78} \]

\[
\frac{F_{el}}{F_{y,i}} = \frac{f_y}{F_{y,i}} \quad \text{(A-2-78)}
\]

\[ F_{el} = \text{elastic local buckling stress of full cross section, ksi (MPa)} \]

\[ \Lambda = \text{upper bound strain limit; for use in Appendix 1, Section 1.2, } \Lambda = 5 \]

In all other cases, \( \Lambda = 15 \).

User Note: The elastic local buckling stress may be obtained numerically or, for I-shaped sections 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 comprising 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:

\[ F_{el} = \frac{k\pi^2 E t^2}{12(1-\nu^2)b^2} \]

For round HSS

\[ F_{el} = \frac{E}{\sqrt{3(1-\nu^2)}} \frac{2t}{D} \]

where
\( b \) = 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 cross section design wall thickness for round HSS, in. (mm)
\( D \) = cross section diameter, in. (mm)
\( \nu \) = 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, \( \phi P_n \), and the allowable CSM tensile strength, \( P_n/\Omega_t \), 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

\[ P_n = f_{csim,t} A_g \quad (A-2-89) \]

\[ \phi_f = 0.90 \text{ (LRFD)} \quad \Omega_t = 1.67 \text{ (ASD)} \]

(b) For tensile rupture in the net section

\[ P_n = F_y A_g \quad (A-2-244) \]

\[ \phi_f = 0.75 \text{ (LRFD)} \quad \Omega_t = 2.00 \text{ (ASD)} \]

where
\( A_g \) = 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)
\( f_{csim,t} \) = stress corresponding to \( \varepsilon_{csim,t} \), ksi (MPa)

\[ \varepsilon_{csim,t} = \frac{F_y + 1.4E_{sh} \varepsilon_y}{E_{sh}} - \varepsilon_y \]

\( (A-2-104) \)

\( \varepsilon_{csim,t} \) = minimum \( \{15\varepsilon_y, \ 0.10(1 - F_y/F_u)\} \) \( (A-2-11) \)
2.5. COMPRESSION STRENGTH

The design CSM compressive strength, $\phi_{pc}$, and the allowable CSM compressive strength, $P_{n}/\Omega_c$, shall be determined as follows:

$$\phi_{pc} = 0.90 \text{ (LRFD)} \quad \Omega_c = 1.67 \text{ (ASD)}$$

The nominal CSM compressive strength at the limit state of yielding or local buckling, $P_n$, is given by:

(a) When $\varepsilon_{cm} / \varepsilon_y < 1.0$

$$P_n = \frac{\varepsilon_{cm}}{\varepsilon_y} F_y A_g$$  \hspace{1cm} (A-2-12)

(b) When $\varepsilon_{cm} / \varepsilon_y \geq 1.0$

$$P_n = f_{cm} A_g$$  \hspace{1cm} (A-2-13)

where

$$f_{cm} = \text{stress corresponding to } \varepsilon_{cm}$$

$$= F_y + E_y \varepsilon_y \left( \frac{\varepsilon_{cm}}{\varepsilon_y} - 1 \right)$$  \hspace{1cm} (A-2-14)

2.6. FLEXURAL STRENGTH

The design CSM flexural strength, $\phi_{pb}$, and the allowable CSM flexural strength, $M_n/\Omega_b$, shall be determined as follows:

$$\phi_{pb} = 0.90 \text{ (LRFD)} \quad \Omega_b = 1.67 \text{ (ASD)}$$

The nominal CSM flexural strength at the limit state of yielding or local buckling, $M_n$, is given by:

(a) I-shaped, channels, tees, HSS, and box section members bent about an axis of symmetry:

(1) When $\varepsilon_{cm} / \varepsilon_y < 1.0$

$$M_n = \frac{\varepsilon_{cm}}{\varepsilon_y} M_y$$  \hspace{1cm} (A-2-15)

(2) When $\varepsilon_{cm} / \varepsilon_y \geq 1.0$

$$M_n = M_p \left[ 1 + \frac{E_y S}{E Z} \left( \frac{\varepsilon_{cm}}{\varepsilon_y} - 1 \right) - \left( 1 - \frac{S}{Z} \right) \left( \frac{\varepsilon_{cm}}{\varepsilon_y} \right)^{\alpha} \right]$$  \hspace{1cm} (A-2-16)

where

$$M_p = F_y Z$$
As an alternative to using Equation A-2-16, the CSM flexural strength is permitted to be obtained by integration of stresses.

(b) 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:

• 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 $\varepsilon_{csm,t}$.

• If $\varepsilon_{csm,max}$ is less than the yield strain $\varepsilon_y$, the nominal CSM flexural strength shall be determined in accordance with Equation A-2-15, with $\varepsilon_{csm} = \varepsilon_{csm,max}$.

• If $\varepsilon_{csm,max}$ is greater than the yield strain $\varepsilon_y$, 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 $\alpha$ taken from Table A-2.2.1.
TABLE A-2.2.1

CSM Bending Coefficient $\alpha$

<table>
<thead>
<tr>
<th>Member Type</th>
<th>Axis of bending</th>
<th>Aspect ratio</th>
<th>$\alpha$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Square and rectangular HSS, and box sections</td>
<td>Any</td>
<td>Any</td>
<td>2.0</td>
</tr>
<tr>
<td>Round HSS</td>
<td>Any</td>
<td>-</td>
<td>2.0</td>
</tr>
<tr>
<td>I-shaped members</td>
<td>major</td>
<td>Any</td>
<td>2.0</td>
</tr>
<tr>
<td></td>
<td>minor</td>
<td>Any</td>
<td>1.2</td>
</tr>
<tr>
<td>Channels</td>
<td>minor</td>
<td>$h/b &lt; 2$</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$h/b \geq 2$</td>
<td>1.0</td>
</tr>
<tr>
<td>Tees</td>
<td>major</td>
<td>$h/b \leq 1$</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$h/b &gt; 1$</td>
<td>1.5</td>
</tr>
<tr>
<td>Equal-leg angle</td>
<td>Any</td>
<td>-</td>
<td>1.0</td>
</tr>
<tr>
<td>Unequal-leg angle</td>
<td>major</td>
<td>Any</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>minor</td>
<td>Any</td>
<td>1.0</td>
</tr>
</tbody>
</table>

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-shaped, 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) I-Shaped, Square, Rectangular and round HSS, and Box Section Members Subject to Flexure and Compression

(1) When $\lambda_i \leq 0.60$

Equations H1-1a and H1-1b shall be satisfied, but with $P_c$ and $M_c$ as defined in this section.

(2) When $\lambda_i > 0.60$

$$\frac{P}{P_c} + \left(\frac{M_{cx}}{M_{cx} + M_{cy}}\right) \leq 1.0 \quad (A-2-17)$$

(b) Round HSS Subject to Flexure and Compression

(1) When $\lambda_i \leq 0.27$

Equations H1-1a and H1-1b shall be satisfied, but with $P_c$ and $M_c$ as defined in this section.

(2) When $\lambda_i > 0.27$

Equation A-2-17 shall be satisfied.

where

$P_c$ = required compressive strength, determined in accordance with Chapter C, using LRFD or ASD load combinations, kips (N)
\( P_c \) = available CSM compressive strength, \( \phi P_c \) or \( P_c/\Omega \), determined in accordance with Section 2.5, kips (N)

\( M_f \) = 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, \( \phi M_c \) or \( M_c/\Omega \), 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.

User Note: See AISC Seismic Provisions for Structural Steel Buildings for structures subject to seismic loads.

The appendix is organized as follows:

3.1. GENERAL PROVISIONS

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

#### 3.3.1 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:

$$F_{SR} = 1,000 \left( \frac{C_f}{n_{SR}} \right)^{0.333} \geq F_{TH} \quad (A-3-1)$$

$$F_{SR} = 6,900 \left( \frac{C_f}{n_{SR}} \right)^{0.333} \geq F_{TH} \quad (A-3-1M)$$

where

- $C_f$ = constant from Table A-3.1 for the fatigue category
- $F_{SR}$ = allowable stress range, ksi (MPa)
- $F_{TH}$ = threshold allowable stress range, maximum stress range for indefinite design life from Table A-3.1, ksi (MPa)
- $n_{SR}$ = number of stress range fluctuations in design life

(b) For stress category F, the allowable stress range, $F_{SR}$, shall be determined by Equation A-3-2 or A-3-2M as follows:
\[ F_{SR} = 100 \left( \frac{1.5}{n_{SR}} \right)^{0.167} \geq 8 \text{ ksi} \]  
(A-3-2)

\[ F_{SR} = 690 \left( \frac{1.5}{n_{SR}} \right)^{0.167} \geq 55 \text{ MPa} \]  
(A-3-2M)

(c) For tension-loaded plate elements connected at their end by cruciform, T
or corner details with 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:

\[ R_{PJP} = \frac{0.65 - 0.59 \left( \frac{2a}{t_p} \right) + 0.72 \left( \frac{w}{t_p} \right)}{t_p^{0.167}} \leq 1.0 \]  
(A-3-4)

\[ R_{PJP} = \frac{1.12 - 1.01 \left( \frac{2a}{t_p} \right) + 1.24 \left( \frac{w}{t_p} \right)}{t_p^{0.167}} \leq 1.0 \]  
(A-3-4M)

where \( R_{PJP} \), the reduction factor for reinforced or nonreinforced trans-
verse PJP groove welds, is determined as follows:

- \( 2a \) = length of the nonwelded root face in the direction of the thick-
- \( t_p \) = design thickness of tension loaded plate, as defined in Section
- \( w \) = 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.

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(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:

$$F_{SR} = 1,000 R_{FIL} \left( \frac{4.4}{n_{SR}} \right)^{0.333} \quad \text{(A-3-5)}$$

$$F_{SR} = 6900 R_{FIL} \left( \frac{4.4}{n_{SR}} \right)^{0.333} \quad \text{(A-3-5M)}$$

where

$$R_{FIL} = \text{reduction factor for joints using a pair of transverse fillet welds only}$$

$$= \frac{0.06 + 0.72 (w/t_p)}{t_p^{0.107}} \leq 1.0 \quad \text{(A-3-6)}$$

$$= \frac{0.103 + 1.24 (w/t_p)}{t_p^{0.107}} \leq 1.0 \quad \text{(A-3-6M)}$$

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 $2 \times 10^8$. Alternatively, if the size of the weld is increased such that $R_{FIL}$ or $R_{PJP}$ is equal to 1.0, then the base metal controls, resulting in stress category C, where there is a fatigue threshold and the crack initiates at the toe of the weld.

### 3.3.2 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 Case 8.5 (stress category G). The net area in tension, $A_t$, is given by Equation A-3-7 or A-3-7M.

$$A_t = \frac{\pi}{4} \left( d_b - \frac{0.9743}{n} \right)^2 \quad \text{(A-3-7)}$$

$$A_t = \frac{\pi}{4} \left( d_b - 0.9382 p \right)^2 \quad \text{(A-3-7M)}$$

where

- $d_b$ = nominal diameter (body or shank diameter), in. (mm)
- $n$ = threads per in. (per mm)
- $p$ = pitch, in. per thread (mm per thread)

For joints in which the material within the grip is not limited to steel or joints that are not tensioned to 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 1/4 in. (6 mm) in size, shall be added at reentrant corners.

The surface roughness of thermally cut edges subject to cyclic stress ranges, that include tension, shall not exceed 1,000 $\mu$m (25 $\mu$m), where Surface Texture, Surface Roughness, Waviness, and Lay (ASME B46.1) is the reference standard.
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.

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.

3.6. 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

<table>
<thead>
<tr>
<th>Description</th>
<th>Stress Category</th>
<th>Constant, ( C_r )</th>
<th>Threshold, ( F_{TH} ) (ksi)</th>
<th>Potential Crack Initiation Point</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>SECTION 1—PLAIN MATERIAL AWAY FROM ANY WELDING</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SECTION 1.1 Base metal, except non-coated weathering steel, with as-rolled or cleaned surfaces; flame cut edges with surface roughness value of 1,000 ( \mu ) in. (25 ( \mu )m) or less, but without reentrant corners</td>
<td>A</td>
<td>25</td>
<td>24 (165)</td>
<td>Away from all welds or structural connections</td>
</tr>
<tr>
<td>SECTION 1.2 Noncoated weathering steel; base metal with as-rolled or cleaned surfaces; flame cut edges with surface roughness value of 1,000 ( \mu ) in. (25 ( \mu )m) or less, but without reentrant corners</td>
<td>B</td>
<td>12</td>
<td>16 (110)</td>
<td>Away from all welds or structural connections</td>
</tr>
<tr>
<td>SECTION 1.23 Members with reentrant corners at copes, cuts, block-outs or other geometrical discontinuities, except weld access holes</td>
<td>C</td>
<td>4.4</td>
<td>10 (69)</td>
<td>At any external edge or at hole perimeter</td>
</tr>
<tr>
<td>( R \geq 1 ) in. (25 mm), with radius, ( R ), formed by predrilling, subpunching and reaming or thermally cut and ground to a bright metal surface</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( R \geq 3/8 ) in. (10 mm) and the radius, ( R ), need not be ground to a bright metal surface</td>
<td>E'</td>
<td>0.39</td>
<td>2.6 (18)</td>
<td></td>
</tr>
<tr>
<td>SECTION 1.34 Rolled cross sections with weld access holes made to requirements of Section J1.6</td>
<td>C</td>
<td>4.4</td>
<td>10 (69)</td>
<td>At reentrant corner of weld access hole</td>
</tr>
<tr>
<td>Access hole ( R \geq 1 ) in. (25 mm) with radius, ( R ), formed by predrilling, subpunching and reaming or thermally cut and ground to a bright metal surface</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Access hole ( R \geq 3/8 ) in. (10 mm) and the radius, ( R ), need not be ground to a bright metal surface</td>
<td>E'</td>
<td>0.39</td>
<td>2.6 (18)</td>
<td></td>
</tr>
</tbody>
</table>
### SECTION 2–CONNECTED MATERIAL IN MECHANICALLY FASTENED JOINTS

<table>
<thead>
<tr>
<th>Description</th>
<th>Area Type</th>
<th>Width</th>
<th>Thickness</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.45 Members with drilled or reamed holes</td>
<td>C</td>
<td>4.4</td>
<td>10 (69)</td>
<td>In net section originating at side of the hole</td>
</tr>
<tr>
<td>Holes containing pretensioned bolts</td>
<td>D</td>
<td>2.2</td>
<td>7 (48)</td>
<td></td>
</tr>
<tr>
<td>Open holes without bolts</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

#### 2.1 Gross area of base metal in lap joints connected by high-strength bolts in joints satisfying all requirements for slip-critical connections

<table>
<thead>
<tr>
<th>Description</th>
<th>Area Type</th>
<th>Width</th>
<th>Thickness</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>Through gross section near hole</td>
<td>B</td>
<td>12</td>
<td>16 (110)</td>
<td></td>
</tr>
</tbody>
</table>

#### 2.2 Base metal at net section of high-strength–bolted joints, designed on the basis of bearing resistance, but fabricated and installed to all requirements for slip-critical connections

<table>
<thead>
<tr>
<th>Description</th>
<th>Area Type</th>
<th>Width</th>
<th>Thickness</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>In net section originating at side of hole</td>
<td>B</td>
<td>12</td>
<td>16 (110)</td>
<td></td>
</tr>
</tbody>
</table>

#### 2.3 Base metal at the net section of riveted joints

<table>
<thead>
<tr>
<th>Description</th>
<th>Area Type</th>
<th>Width</th>
<th>Thickness</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>In net section originating at side of hole</td>
<td>G</td>
<td>4.4</td>
<td>10 (69)</td>
<td></td>
</tr>
</tbody>
</table>

#### 2.34 Base metal at net section of eyebar head or pin plate

<table>
<thead>
<tr>
<th>Description</th>
<th>Area Type</th>
<th>Width</th>
<th>Thickness</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>In net section originating at side of hole</td>
<td>E</td>
<td>1.1</td>
<td>4.5 (31)</td>
<td></td>
</tr>
</tbody>
</table>
TABLE A-3.1 (continued)
Fatigue Design Parameters

<table>
<thead>
<tr>
<th>1.1 and 1.2</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a)</td>
</tr>
<tr>
<td>(b)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>1.3</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a) R</td>
</tr>
<tr>
<td>(b) R</td>
</tr>
<tr>
<td>(c) R</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>1.4</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>1.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a) As seen with bracing removed</td>
</tr>
<tr>
<td>(b)</td>
</tr>
</tbody>
</table>
2.1 As seen with lap plate removed
(a) (b) (c)
(Note: Figures are for slip-critical bolted connections.)

2.2 As seen with lap plate removed
(a) (b) (c)
(Note: Figures are for bolted connections designed to bear, meeting the requirements of slip-critical connections.)

2.3 As seen with lap plate removed
(a) (b) (c)
(Note: Figures are for snug-tightened bolts, rivets, or other mechanical fasteners.)

2.4
### TABLE A-3.1 (continued)
**Fatigue Design Parameters**

<table>
<thead>
<tr>
<th>Description</th>
<th>Stress Category</th>
<th>Constant, $C_t$</th>
<th>Threshold, $F_{th}$, ksi (MPa)</th>
<th>Potential Crack Initiation Point</th>
</tr>
</thead>
<tbody>
<tr>
<td>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</td>
<td>B</td>
<td>12</td>
<td>16 (110)</td>
<td>From surface or internal discontinuities in weld</td>
</tr>
<tr>
<td>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</td>
<td>$B'$</td>
<td>6.1</td>
<td>12 (83)</td>
<td>From surface or internal discontinuities in weld</td>
</tr>
<tr>
<td>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</td>
<td>D</td>
<td>2.2</td>
<td>7 (48)</td>
<td>From the weld termination into the web or flange</td>
</tr>
<tr>
<td>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</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Access hole $R \geq 3/8$ in. (10 mm) and the radius, $R$, need not be ground to a bright metal surface</td>
<td>E'</td>
<td>0.39</td>
<td>2.6 (18)</td>
<td></td>
</tr>
<tr>
<td>3.4 Base metal at ends of longitudinal intermittent fillet weld segments</td>
<td>E</td>
<td>1.1</td>
<td>4.5 (31)</td>
<td>In connected material at start and stop locations of any weld</td>
</tr>
<tr>
<td>3.5 Base metal at ends of partial length welded coverplates narrower than the flange having square or tapered ends, with or without welds across the ends</td>
<td>$t_f \leq 0.8$ in. (20 mm)</td>
<td>$E$</td>
<td>4.4</td>
<td>4.5 (31)</td>
</tr>
<tr>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td></td>
<td>$t_f &gt; 0.8$ in. (20 mm)</td>
<td>$E'$</td>
<td>0.39</td>
<td>2.6 (18)</td>
</tr>
<tr>
<td>3.6 Base metal at ends of partial length welded coverplates or other attachments wider than the flange with welds across the ends</td>
<td>$t_f \leq 0.8$ in. (20 mm)</td>
<td>$E$</td>
<td>1.1</td>
<td>4.5 (31)</td>
</tr>
<tr>
<td></td>
<td>$t_f &gt; 0.8$ in. (20 mm)</td>
<td>$E'$</td>
<td>0.39</td>
<td>2.6 (18)</td>
</tr>
<tr>
<td>3.7 Base metal at ends of partial length welded coverplates wider than the flange without welds across the ends</td>
<td>$t_f \leq 0.8$ in. (20 mm)</td>
<td>$E'$</td>
<td>0.39</td>
<td>2.6 (18)</td>
</tr>
<tr>
<td></td>
<td>$t_f &gt; 0.8$ in. (20 mm) is not permitted</td>
<td>None</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

**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 | $t \leq 0.5$ in. (13 mm) | $E$ | 1.1 | 4.5 (31) | Initiating from end of any weld termination extending into the base metal |
| | $t > 0.5$ in. (13 mm) | $E'$ | 0.39 | 2.6 (18) | |
3.1

(a)  

(b)  

(c)  

(d)  

(e)  

3.2

(a)  

(b)  

(c)  

(d)  

(e)  

3.3

(a)  

(b)  

3.4

(a)  

(b)  

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3.5

(a)  
(b)  
(c)  

3.6

(a)  
(b)  
(c)  

3.7

(a) No weld  
(b) Typ.  

4.1

(a)  
(b)  

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### TABLE A-3.1  (continued)

**Fatigue Design Parameters**

<table>
<thead>
<tr>
<th>Description</th>
<th>Stress Category</th>
<th>Constant, ( C_i )</th>
<th>Threshold, ( F_{TH} ), ksi (MPa)</th>
<th>Potential Crack Initiation Point</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>SECTION 5–WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.1 Weld metal and base metal in or adjacent to CJP grove welded splices in</td>
<td>B</td>
<td>12</td>
<td>16 (110)</td>
<td>From internal discontinuities in</td>
</tr>
<tr>
<td>plate, rolled shapes, or built-up cross sections with no change in cross</td>
<td></td>
<td></td>
<td></td>
<td>weld metal or along the fusion</td>
</tr>
<tr>
<td>section with welds ground essentially parallel to the direction of stress</td>
<td></td>
<td></td>
<td></td>
<td>boundary</td>
</tr>
<tr>
<td>and inspected in accordance with Section 3.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( F_{y} &lt; 90 \text{ ksi (620 MPa)} )</td>
<td>B</td>
<td>12</td>
<td>16 (110)</td>
<td>From internal discontinuities in</td>
</tr>
<tr>
<td>( F_{y} \geq 90 \text{ ksi (620 MPa)} )</td>
<td>B'</td>
<td>8.1</td>
<td>12 (83)</td>
<td>metal or along the fusion</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>boundary at start of transition</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>when ( F_{y} &gt; 90 \text{ ksi (620 MPa)} )</td>
</tr>
<tr>
<td>5.2 Weld metal and base metal in or adjacent to CJP grove welded splices</td>
<td></td>
<td></td>
<td></td>
<td>From internal discontinuities in</td>
</tr>
<tr>
<td>with welds ground essentially parallel to the direction of stress at</td>
<td></td>
<td></td>
<td></td>
<td>metal or along the fusion</td>
</tr>
<tr>
<td>transitions in thickness or width made on a slope no greater than 1:2-1/2</td>
<td></td>
<td></td>
<td></td>
<td>boundary at start of transition</td>
</tr>
<tr>
<td>and inspected in accordance with Section 3.6</td>
<td></td>
<td></td>
<td></td>
<td>when ( F_{y} &gt; 90 \text{ ksi (620 MPa)} )</td>
</tr>
<tr>
<td>( F_{y} &lt; 90 \text{ ksi (620 MPa)} )</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( F_{y} \geq 90 \text{ ksi (620 MPa)} )</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.3 Base metal and weld metal in or adjacent to CJP grove welded splices</td>
<td></td>
<td></td>
<td></td>
<td>From internal discontinuities in</td>
</tr>
<tr>
<td>with welds ground essentially parallel to the direction of stress at</td>
<td></td>
<td></td>
<td></td>
<td>weld metal or along the fusion</td>
</tr>
<tr>
<td>transitions in width made on a radius, ( R ), of not less than 24 in. (</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(600 mm) with the point of tangency at the end of the groove weld and</td>
<td></td>
<td></td>
<td></td>
<td>boundary</td>
</tr>
<tr>
<td>inspected in accordance with Section 3.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.4 Weld metal and base metal in or adjacent to CJP grove welds in T- or</td>
<td>C</td>
<td>4.4</td>
<td>10 (69)</td>
<td>From weld extending into base</td>
</tr>
<tr>
<td>corner-joints or splices, without transitions in thickness or with</td>
<td></td>
<td></td>
<td></td>
<td>metal or into weld metal</td>
</tr>
<tr>
<td>transition in thickness having slopes no greater than 1:2-1/2, when weld</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>reinforcement is not removed, and is inspected in accordance with Section 3.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.5 Base metal and weld metal in or adjacent to transverse CJP groove welded butt splices with backing left in place</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>---</td>
<td>---</td>
<td>---</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tack welds inside groove</td>
<td>D</td>
<td>2.2</td>
<td>7 (48)</td>
<td></td>
</tr>
<tr>
<td>Tack welds outside the groove and not closer than 1/2 in. (13 mm) to the edge of base metal</td>
<td>E</td>
<td>1.1</td>
<td>4.5 (31)</td>
<td></td>
</tr>
</tbody>
</table>

| 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; $F_{S0k}$ shall be the smaller of the toe crack or root crack allowable stress range |
|---|---|---|
| Crack initiating from weld toe | C | 4.4 | 10 (69) |
| Crack initiating from weld root | C' | See Eq. A-3-3 or A-3-3M | None |

From the toe of the groove weld or the toe of the weld attaching backing when applicable
TABLE A-3.1 (continued)
Fatigue Design Parameters

<table>
<thead>
<tr>
<th>5.1</th>
<th><img src="a" alt="Diagram" /></th>
<th><img src="b" alt="Diagram" /></th>
</tr>
</thead>
<tbody>
<tr>
<td>5.2</td>
<td><img src="a" alt="Diagram" /></td>
<td><img src="b" alt="Diagram" /></td>
</tr>
<tr>
<td>5.3</td>
<td><img src="a" alt="Diagram" /></td>
<td><img src="b" alt="Diagram" /></td>
</tr>
<tr>
<td>5.4</td>
<td><img src="a" alt="Diagram" /></td>
<td><img src="b" alt="Diagram" /></td>
</tr>
</tbody>
</table>
### TABLE A-3.1 (continued)  
Fatigue Design Parameters

<table>
<thead>
<tr>
<th>Description</th>
<th>Stress Category</th>
<th>Constant ( C_r )</th>
<th>Threshold ( F_{th} ), ksi (MPa)</th>
<th>Potential Crack Initiation Point</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>SECTION 5–WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS</strong> (cont’d)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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; \( F_{th} \) 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

<table>
<thead>
<tr>
<th>Condition</th>
<th>( R \geq 24 \text{ in. (600 mm)} )</th>
<th>( 24 \text{ in.} \leq R &lt; 24 \text{ in.} ) ((150 \text{ mm} \leq R &lt; 600 \text{ mm}))</th>
<th>( 6 \text{ in.} \leq R &lt; 24 \text{ in.} ) ((150 \text{ mm} \leq R &lt; 600 \text{ mm}))</th>
<th>( 2 \text{ in.} \leq R &lt; 6 \text{ in.} ) ((50 \text{ mm} \leq R &lt; 150 \text{ mm}))</th>
<th>( R &lt; 2 \text{ in.} ) ((50 \text{ mm}))</th>
</tr>
</thead>
<tbody>
<tr>
<td>( B )</td>
<td>12</td>
<td>16 (110)</td>
<td></td>
<td></td>
<td>Near point of tangency of radius at edge of member</td>
</tr>
<tr>
<td>( C )</td>
<td>4.4</td>
<td>10 (69)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( D )</td>
<td>2.2</td>
<td>7 (48)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( E )</td>
<td>1.1</td>
<td>4.5 (31)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### TABLE A-3.1 (continued)
#### Fatigue Design Parameters

<table>
<thead>
<tr>
<th>5.7</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a)</td>
</tr>
<tr>
<td>(b)</td>
</tr>
<tr>
<td>(c)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>5.8</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a)</td>
</tr>
<tr>
<td>(b)</td>
</tr>
<tr>
<td>(c)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>6.1</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a)</td>
</tr>
<tr>
<td>(b)</td>
</tr>
<tr>
<td>(c)</td>
</tr>
</tbody>
</table>
### TABLE A-3.1 (continued)
**Fatigue Design Parameters**

<table>
<thead>
<tr>
<th>Description</th>
<th>Stress Category</th>
<th>Constant $C_f$</th>
<th>Threshold $F_{TH}$, ksi (MPa)</th>
<th>Potential Crack Initiation Point</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>SECTION 6–BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (continued)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>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</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(a) When weld reinforcement is removed</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>24 in. (600 mm)</td>
<td>$B$ 12</td>
<td>16 (110)</td>
<td></td>
<td>Near point of tangency of radius or in the weld or at fusion boundary or member or attachment</td>
</tr>
<tr>
<td>$\leq R &lt; 24$ in.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(150 mm $\leq R &lt; 600$ mm)</td>
<td>$C$ 4.4</td>
<td>10 (69)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 in. $\leq R &lt; 6$ in.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(50 mm $\leq R &lt; 150$ mm)</td>
<td>$D$ 2.2</td>
<td>7 (48)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$R &lt; 2$ in. (50 mm)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(b) When weld reinforcement is not removed</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$R \geq 6$ in. (150 mm)</td>
<td>$C$ 4.4</td>
<td>10 (69)</td>
<td></td>
<td>At toe of the weld either along edge of member or the attachment</td>
</tr>
<tr>
<td>2 in. $\leq R &lt; 6$ in.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(50 mm $\leq R &lt; 150$ mm)</td>
<td>$D$ 2.2</td>
<td>7 (48)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$R &lt; 2$ in. (50 mm)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.3 Base metal at details of unequal thickness attached by CJP groove welds, subject to</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
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:

<table>
<thead>
<tr>
<th>( R )</th>
<th>D</th>
<th>E</th>
</tr>
</thead>
<tbody>
<tr>
<td>( R &gt; 2 \text{ in. (50 mm)} )</td>
<td>2.2</td>
<td>7 (48)</td>
</tr>
<tr>
<td>( R \leq 2 \text{ in. (50 mm)} )</td>
<td>1.1</td>
<td>4.5 (31)</td>
</tr>
</tbody>
</table>

*When reinforcement is not removed*

<table>
<thead>
<tr>
<th>( R )</th>
<th>D</th>
<th>E</th>
</tr>
</thead>
<tbody>
<tr>
<td>( R &gt; 2 \text{ in. (50 mm)} )</td>
<td>2.2</td>
<td></td>
</tr>
<tr>
<td>( R \leq 2 \text{ in. (50 mm)} )</td>
<td>1.1</td>
<td></td>
</tr>
</tbody>
</table>

*At toe of weld along edge of thinner material*

*In weld termination in small radius*

*At toe of weld along edge of thinner material*
TABLE A-3.1 (continued)
Fatigue Design Parameters

6.2

(a)
(b)
(c)

6.3

CJP, Ends ground smooth
R

CJP w/ reinforcement
Ends ground smooth
R

(d)
(e)

CJP, Ends ground smooth
R

Grind
TABLE A-3.1 (continued)
Fatigue Design Parameters

<table>
<thead>
<tr>
<th>Description</th>
<th>Constant $C_f$</th>
<th>Threshold $F_{th}$, ksi (MPa)</th>
<th>Potential Crack Initiation Point</th>
</tr>
</thead>
<tbody>
<tr>
<td>SECTION 6–BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (continued)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>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 J-p groove welds parallel to direction of stress when the detail embodies a transition radius, $R$, with weld termination ground smooth</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$R &gt; 2$ in. (50 mm)</td>
<td>D</td>
<td>2.2</td>
<td>Initiating in base metal at the weld termination or at the toe of the weld extending into the base metal</td>
</tr>
<tr>
<td>$R \leq 2$ in. (50 mm)</td>
<td>E</td>
<td>1.1</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>4.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(31)</td>
<td></td>
</tr>
</tbody>
</table>
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, \( b \):  

<table>
<thead>
<tr>
<th>Condition</th>
<th>( a &lt; 2 \text{ in. (50 mm)} ) for any thickness, ( b )</th>
<th>( 2 \text{ in. (50 mm)} \leq a \leq \text{lesser of } 12b \text{ or 4 in. (100 mm)} )</th>
<th>( a &gt; \text{lesser of } 12b \text{ or 4 in. (100 mm)} ) when ( b \leq 0.8 \text{ in. (20 mm)} )</th>
<th>( a &gt; \text{lesser of } 12b \text{ or 4 in. (100 mm)} ) when ( b &gt; 0.8 \text{ in. (20 mm)} )</th>
<th>Initiating in base metal at the weld termination or at the toe of the weld extending into the base metal</th>
</tr>
</thead>
<tbody>
<tr>
<td>( C )</td>
<td>4.4</td>
<td>10 (69)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( D )</td>
<td>2.2</td>
<td>7 (48)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( E )</td>
<td>1.1</td>
<td>4.5 (31)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( E' )</td>
<td>0.39</td>
<td>2.6 (18)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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:

<table>
<thead>
<tr>
<th>Condition</th>
<th>( R &gt; 2 \text{ in. (50 mm)} )</th>
<th>( R \leq 2 \text{ in. (50 mm)} )</th>
<th>Initiating in base metal at the weld termination, extending into the base metal</th>
</tr>
</thead>
<tbody>
<tr>
<td>( D )</td>
<td>2.2</td>
<td>7 (48)</td>
<td></td>
</tr>
<tr>
<td>( E )</td>
<td>1.1</td>
<td>4.5 (31)</td>
<td></td>
</tr>
</tbody>
</table>

[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**

**6.4**

<table>
<thead>
<tr>
<th>(a)</th>
<th>(b)</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image1" alt="Diagram (a)" /></td>
<td><img src="image2" alt="Diagram (b)" /></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>(c)</th>
<th>(d)</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image3" alt="Diagram (c)" /></td>
<td><img src="image4" alt="Diagram (d)" /></td>
</tr>
</tbody>
</table>

**7.1**

<table>
<thead>
<tr>
<th>(a)</th>
<th>(b)</th>
<th>(c)</th>
<th>(d)</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image5" alt="Diagram (a)" /></td>
<td><img src="image6" alt="Diagram (b)" /></td>
<td><img src="image7" alt="Diagram (c)" /></td>
<td><img src="image8" alt="Diagram (d)" /></td>
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</tbody>
</table>

**7.2**

<table>
<thead>
<tr>
<th>(a)</th>
<th>(b)</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image9" alt="Diagram (a)" /></td>
<td><img src="image10" alt="Diagram (b)" /></td>
</tr>
</tbody>
</table>
### TABLE A-3.1 (continued)
Fatigue Design Parameters

<table>
<thead>
<tr>
<th>Description</th>
<th>Stress Category</th>
<th>Constant $C_f$</th>
<th>Threshold $k_f$ (ksi)</th>
<th>Potential Crack Initiation Point</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>SECTION 8–MISCELLANEOUS</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8.11 Base metal at steel headed stud anchors attached by fillet weld or automatic stud welding</td>
<td>C</td>
<td>4.4</td>
<td>10 (69)</td>
<td>At toe of weld in base metal</td>
</tr>
<tr>
<td>8.12 Shear on throat of any fillet weld, continuous or intermittent, longitudinal or transverse</td>
<td>F</td>
<td>See Eq. A-3-2 or A-3-2M</td>
<td>See Eq. A-3-2 or A-3-2M</td>
<td>Initiating at the root of the fillet weld, extending into the weld</td>
</tr>
<tr>
<td>8.23 Base metal at plug or slot welds</td>
<td>E</td>
<td>1.1</td>
<td>4.5 (31)</td>
<td>Initiating in the base metal at the end of the plug or slot weld, extending into the base metal</td>
</tr>
<tr>
<td>8.34 Shear on plug or slot welds</td>
<td>F</td>
<td>See Eq. A-3-2 or A-3-2M</td>
<td>See Eq. A-3-2 or A-3-2M</td>
<td>Initiating in the weld at the faying surface, extending into the weld</td>
</tr>
<tr>
<td>8.4 High-strength bolts, common bolts, threaded anchor rods, and hanger rods, whether pretensioned in accordance with Equation J3.5-Table J3.1 or J3.1M, or snug-tightened with cut, ground or rolled threads; stress range on tensile stress area due to applied cyclic load plus prying action, when applicable</td>
<td>G</td>
<td>0.39</td>
<td>7 (48)</td>
<td>Initiating at the root of the threads, extending into the fastener</td>
</tr>
</tbody>
</table>
### TABLE A-3.1 (continued)
**Fatigue Design Parameters**

<table>
<thead>
<tr>
<th>Column 1</th>
<th>Column 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a)</td>
<td>(b)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Column 1</th>
<th>Column 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a)</td>
<td>(b)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Column 1</th>
<th>Column 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a)</td>
<td>(b)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Column 1</th>
<th>Column 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a)</td>
<td>(b)</td>
</tr>
</tbody>
</table>
APPENDIX 4

STRUCTURAL DESIGN FOR FIRE CONDITIONS

This appendix provides criteria for the design and evaluation of structural 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 materials at elevated temperatures that cause progressive decrease in strength and stiffness of structural components and systems at elevated temperatures.

User Note: Throughout this chapter, the term “elevated temperatures” refers to temperatures due to unintended fire exposure only.

The appendix is organized as follows:

4.2. Structural Design for Fire Conditions by Analysis
4.3. Design by Qualification Testing

4.1. GENERAL PROVISIONS

The methods contained in this appendix provide regulatory evidence of compliance in accordance with the design applications outlined in this section.

1. Performance Objective

Structural components, members, and building frame systems shall be designed so as to maintain their load-bearing function during the design-basis fire and to satisfy other performance requirements specified for the building occupancy.

Deformation criteria shall be applied where the means of providing structural fire resistance, or the design criteria for fire barriers, requires evaluation of the deformation of the load-carrying structure.

Within the compartment of fire origin, forces and deformations from the design-basis fire shall not cause a breach of horizontal or vertical compartmentation.

2. Design by Engineering Analysis

The analysis methods in Section 4.2 are permitted to be used to document the anticipated performance of steel framing when subjected to design-basis fire scenarios. Methods in Section 4.2 provide evidence of compliance with performance objectives established in Section 4.1.1.

The analysis methods in Section 4.2 are permitted to be used to demonstrate an equivalency for an alternative material or method, as permitted by the applicable building code (ABC).
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.

An 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, \(D, L, \text{ and } S\) 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 1800°F (20 to 980°C) shall be taken as follows:

- Austenitic stainless steels: $11 \times 10^{-6}/°F (1.9 \times 10^{-5}/°C)$.
- Duplex stainless steels: $9.1 \times 10^{-6}/°F (1.6 \times 10^{-5}/°C)$.

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 $F_r(T)$, $F_y(T)$, $F_2(T)$ 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-4.2.2 to Table A-4.2.5. 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 7.2.
### TABLE A-4.2.2
Properties of Austenitic Type S30400/S30403 at Elevated Temperatures

<table>
<thead>
<tr>
<th>Steel Temperature, °F (°C)</th>
<th>$k_E = E(T)/E_0$</th>
<th>$k_y = F_y(T)/F_y$</th>
<th>$k_2 = F_2(T)/F_y$</th>
<th>$k_a = F_a(T)/F_a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>68 (20)</td>
<td>1.00</td>
<td>1.00</td>
<td>1.31</td>
<td>1.00</td>
</tr>
<tr>
<td>200 (93)</td>
<td>0.96</td>
<td>0.80</td>
<td>1.05</td>
<td>0.83</td>
</tr>
<tr>
<td>400 (200)</td>
<td>0.92</td>
<td>0.65</td>
<td>0.88</td>
<td>0.72</td>
</tr>
<tr>
<td>600 (320)</td>
<td>0.87</td>
<td>0.59</td>
<td>0.81</td>
<td>0.68</td>
</tr>
<tr>
<td>750 (400)</td>
<td>0.84</td>
<td>0.55</td>
<td>0.78</td>
<td>0.66</td>
</tr>
<tr>
<td>800 (430)</td>
<td>0.83</td>
<td>0.54</td>
<td>0.77</td>
<td>0.65</td>
</tr>
<tr>
<td>1000 (540)</td>
<td>0.78</td>
<td>0.48</td>
<td>0.71</td>
<td>0.58</td>
</tr>
<tr>
<td>1200 (650)</td>
<td>0.74</td>
<td>0.42</td>
<td>0.61</td>
<td>0.47</td>
</tr>
<tr>
<td>1400 (760)</td>
<td>0.66</td>
<td>0.30</td>
<td>0.43</td>
<td>0.31</td>
</tr>
<tr>
<td>1600 (870)</td>
<td>0.50</td>
<td>0.18</td>
<td>0.23</td>
<td>0.16</td>
</tr>
<tr>
<td>1800 (980)</td>
<td>0.24</td>
<td>0.09</td>
<td>0.10</td>
<td>0.09</td>
</tr>
<tr>
<td>2000 (1100)</td>
<td>0.11</td>
<td>0.05</td>
<td>0.06</td>
<td>0.05</td>
</tr>
</tbody>
</table>

### TABLE A-4.2.3
Properties of Austenitic Type S31600/S31603 at Elevated Temperatures

<table>
<thead>
<tr>
<th>Steel Temperature, °F (°C)</th>
<th>$k_E = E(T)/E_0$</th>
<th>$k_y = F_y(T)/F_y$</th>
<th>$k_2 = F_2(T)/F_y$</th>
<th>$k_a = F_a(T)/F_a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>68 (20)</td>
<td>1.00</td>
<td>1.00</td>
<td>1.19</td>
<td>1.00</td>
</tr>
<tr>
<td>200 (93)</td>
<td>0.96</td>
<td>0.87</td>
<td>1.14</td>
<td>0.88</td>
</tr>
<tr>
<td>400 (200)</td>
<td>0.92</td>
<td>0.72</td>
<td>0.98</td>
<td>0.80</td>
</tr>
<tr>
<td>600 (320)</td>
<td>0.87</td>
<td>0.66</td>
<td>0.91</td>
<td>0.78</td>
</tr>
<tr>
<td>750 (400)</td>
<td>0.84</td>
<td>0.62</td>
<td>0.85</td>
<td>0.77</td>
</tr>
<tr>
<td>800 (430)</td>
<td>0.83</td>
<td>0.61</td>
<td>0.84</td>
<td>0.76</td>
</tr>
<tr>
<td>1000 (540)</td>
<td>0.78</td>
<td>0.58</td>
<td>0.79</td>
<td>0.71</td>
</tr>
<tr>
<td>1200 (650)</td>
<td>0.74</td>
<td>0.53</td>
<td>0.72</td>
<td>0.59</td>
</tr>
<tr>
<td>1400 (760)</td>
<td>0.66</td>
<td>0.45</td>
<td>0.57</td>
<td>0.41</td>
</tr>
<tr>
<td>1600 (870)</td>
<td>0.50</td>
<td>0.27</td>
<td>0.33</td>
<td>0.23</td>
</tr>
<tr>
<td>1800 (980)</td>
<td>0.24</td>
<td>0.15</td>
<td>–</td>
<td>0.12</td>
</tr>
<tr>
<td>2000 (1100)</td>
<td>0.11</td>
<td>0.07</td>
<td>–</td>
<td>0.07</td>
</tr>
</tbody>
</table>
### TABLE A-4.2.4
Properties of Duplex Types S32202 and S32304 at Elevated Temperatures

<table>
<thead>
<tr>
<th>Steel Temperature, °F (°C)</th>
<th>$k_E = E(T)/E$</th>
<th>$k_y = F_y(T)/F_y$</th>
<th>$k_z = F_z(T)/F_z$</th>
<th>$k_a = F_a(T)/F_a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>68 (20)</td>
<td>1.00</td>
<td>1.00</td>
<td>1.15</td>
<td>1.00</td>
</tr>
<tr>
<td>200 (93)</td>
<td>0.96</td>
<td>0.84</td>
<td>0.96</td>
<td>0.95</td>
</tr>
<tr>
<td>400 (200)</td>
<td>0.92</td>
<td>0.75</td>
<td>0.82</td>
<td>0.87</td>
</tr>
<tr>
<td>600 (320)</td>
<td>0.87</td>
<td>0.67</td>
<td>0.76</td>
<td>0.78</td>
</tr>
<tr>
<td>750 (400)</td>
<td>0.84</td>
<td>0.58</td>
<td>0.70</td>
<td>0.70</td>
</tr>
<tr>
<td>800 (430)</td>
<td>0.83</td>
<td>0.54</td>
<td>0.67</td>
<td>0.67</td>
</tr>
<tr>
<td>1000 (540)</td>
<td>0.78</td>
<td>0.37</td>
<td>0.53</td>
<td>0.54</td>
</tr>
<tr>
<td>1200 (650)</td>
<td>0.74</td>
<td>0.21</td>
<td>0.37</td>
<td>0.40</td>
</tr>
<tr>
<td>1400 (760)</td>
<td>0.66</td>
<td>0.10</td>
<td>0.20</td>
<td>0.25</td>
</tr>
<tr>
<td>1600 (870)</td>
<td>0.50</td>
<td>0.05</td>
<td>0.08</td>
<td>0.12</td>
</tr>
<tr>
<td>1800 (980)</td>
<td>0.24</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>2000 (1100)</td>
<td>0.11</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
</tbody>
</table>

### TABLE A-4.2.5
Properties of Duplex Types S32003, S32101, S32205, S82011, and S82441 at Elevated Temperatures

<table>
<thead>
<tr>
<th>Steel Temperature, °F (°C)</th>
<th>$k_E = E(T)/E$</th>
<th>$k_y = F_y(T)/F_y$</th>
<th>$k_z = F_z(T)/F_z$</th>
<th>$k_a = F_a(T)/F_a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>68 (20)</td>
<td>1.00</td>
<td>1.00</td>
<td>1.12</td>
<td>1.00</td>
</tr>
<tr>
<td>200 (93)</td>
<td>0.98</td>
<td>0.84</td>
<td>0.97</td>
<td>0.96</td>
</tr>
<tr>
<td>400 (200)</td>
<td>0.92</td>
<td>0.70</td>
<td>0.86</td>
<td>0.91</td>
</tr>
<tr>
<td>600 (320)</td>
<td>0.87</td>
<td>0.64</td>
<td>0.81</td>
<td>0.87</td>
</tr>
<tr>
<td>750 (400)</td>
<td>0.84</td>
<td>0.60</td>
<td>0.76</td>
<td>0.82</td>
</tr>
<tr>
<td>800 (430)</td>
<td>0.83</td>
<td>0.58</td>
<td>0.73</td>
<td>0.79</td>
</tr>
<tr>
<td>1000 (540)</td>
<td>0.78</td>
<td>0.49</td>
<td>0.62</td>
<td>0.65</td>
</tr>
<tr>
<td>1200 (650)</td>
<td>0.74</td>
<td>0.35</td>
<td>0.46</td>
<td>0.47</td>
</tr>
<tr>
<td>1400 (760)</td>
<td>0.66</td>
<td>0.20</td>
<td>0.27</td>
<td>0.28</td>
</tr>
<tr>
<td>1600 (870)</td>
<td>0.50</td>
<td>0.09</td>
<td>0.14</td>
<td>0.16</td>
</tr>
<tr>
<td>1800 (980)</td>
<td>0.24</td>
<td>0.02</td>
<td>0.05</td>
<td>0.07</td>
</tr>
<tr>
<td>2000 (1100)</td>
<td>0.11</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
</tbody>
</table>

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.

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Public Review draft dated October 14, 2020
AMERICAN INSTITUTE OF STEEL CONSTRUCTION
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-resistant materials, as per Section 4.2.2.

The mechanical response results in forces and deformations in the structural system subjected to the thermal response calculated from the design-basis fire. The mechanical response shall take into account explicitly the deterioration in strength and stiffness with increasing temperature, the effects of thermal expansions, 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 member 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 (f).

(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 structural 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.

\[
F_{cr} (T) = 1.2 \left( \frac{E(T)}{F_y (T)} \right) \left( \frac{F_y (T)}{F_v (T)} \right)^{\alpha(T)} \left( \frac{F_y (T)}{F_v (T)} \right)^{\gamma(T)} \]

\[ \leq F_y (T) \]

(A-4-2)

(1) When \( \frac{L_e}{r} \leq 5.62 \left( \frac{E(T)}{F_y (T)} \right) \left( \frac{F_y (T)}{F_v (T)} \right) \leq 3.20 \)

\[
F_{cr} (T) = 1.2 \left( \frac{E(T)}{F_y (T)} \right) \left( \frac{F_y (T)}{F_v (T)} \right)^{\alpha(T)} \left( \frac{F_y (T)}{F_v (T)} \right)^{\gamma(T)} \]

(A-4-2)

(2) When \( \frac{L_e}{r} > 5.62 \left( \frac{E(T)}{F_y (T)} \right) \left( \frac{F_y (T)}{F_v (T)} \right) \) or \( \frac{F_y (T)}{F_v (T)} > 3.20 \)

\[
F_{cr} (T) = \beta_2 (T) F_v (T) \]

(A-4-3)

where

\( E(T) \) = elastic modulus at elevated temperature determined using the coefficients from Table A-4-2-1
\( F_y (T) \) = yield stress at elevated temperature determined using the coefficients from Table A-4-2-1
\( F_v (T) \) = 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-4.2.6
### Table A-4.2.6
Flexural Buckling Coefficients at elevated temperatures

<table>
<thead>
<tr>
<th>Member type</th>
<th>Alloy family</th>
<th>$\alpha(T)$</th>
<th>$\beta_\delta(T)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rolled or built-up I-shaped sections buckling about the minor axis, and other sections not specified in this table</td>
<td>Austenitic And Duplex</td>
<td>0.55</td>
<td>$0.75 - 0.25 \left(1 - \frac{k_x}{k_E}\right)$</td>
</tr>
<tr>
<td>Rolled or built-up I-shaped sections buckling about the major axis, welded box sections, and round HSS</td>
<td>Austenitic And Duplex</td>
<td>0.58</td>
<td>$0.82 - 0.40 \left(1 - \frac{k_y}{k_E}\right)$</td>
</tr>
<tr>
<td>Rectangular HSS</td>
<td>Austenitic And Duplex</td>
<td>0.69 - 0.30</td>
<td>$0.82 - 0.10 \left(1 - \frac{k_y}{k_E}\right)$</td>
</tr>
</tbody>
</table>

(c) Design for Flexure

For 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 lateraltorsional 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 structural stainless steel properties as stipulated in Section 4.2.3b and using the lateral-torsional buckling coefficient at elevated temperature given in Table A-4.2.7 in lieu of the lateral-torsional buckling coefficients given in Table F2.1.

### Table A-4.2.7
Lateral-Torsional Buckling Coefficients at elevated temperatures

<table>
<thead>
<tr>
<th>Alloy Family</th>
<th>$\beta_{LT}(T)$</th>
<th>$\beta_{\delta LT}(T)$</th>
<th>$\beta_{\phi LT}(T)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Austenitic</td>
<td>$0.82 - 0.10 \left(1 - \frac{k_x}{k_E}\right)$</td>
<td>$0.90 - 0.11 \left(1 - \frac{k_y}{k_E}\right)$</td>
<td>0.40</td>
</tr>
<tr>
<td>Duplex</td>
<td>$0.86 - 0.10 \left(1 - \frac{k_y}{k_E}\right)$</td>
<td>$1.10 - 0.13 \left(1 - \frac{k_y}{k_E}\right)$</td>
<td>0.50</td>
</tr>
</tbody>
</table>

The material properties at elevated temperatures, $E(T)$ and $F_y(T)$, and the $k_x$ and $k_y$ coefficients are calculated in accordance with Tables A-4.2.2 to A-4.2.5, 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, 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).

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 A3.1d are critical to the performance of the structure, the Charpy V-notch toughness shall be determined in accordance with the provisions of Section A3.1d. If the notch toughness so determined does not meet the provisions of Section A3.1d, the EOR shall determine if remedial actions are required.

5. **Weld Metal**

Where structural performance is dependent on existing welded connections, representative samples of weld metal shall be obtained. Chemical analysis and mechanical tests shall be made to characterize the weld metal. A determination shall be made of the magnitude and consequences of imperfections. If the requirements of 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 and a tensile strength of 65 ksi 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.

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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, $L_r, S, R$ shall be substituted for $L$.

where

- $L_r = \text{nominal roof live load}$
- $R = \text{nominal load due to rainwater or snow, exclusive of the ponding contribution}$
- $S = \text{nominal snow load}$

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 carbon 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 at the braced point in a column, beam or beam-column.

The appendix is organized as follows:

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 to 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 to 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 (that is, the lateral displacement of one end of the segment relative to the other). A point brace (formerly referred to as a nodal brace) controls the movement at the braced point without direct interaction with adjacent braced points. A continuous bracing system consists of bracing that is attached along the entire member length.

The available strength and stiffness of the bracing members and connections shall equal or exceed the required strength and stiffness, respectively, unless analysis indicates that smaller values are justified.

Columns, beams and beam-columns with end and intermediate braced points designed to meet the requirements in Sections 6.2, 6.3 and 6.4, as applicable, are permitted to be designed based on lengths $L_e$ and $L_s$, 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 $2/\phi \times (L_{RFD})$ or $2 \Omega_{ASD}$ 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, $L_e$ and $L_b$, taken less than the distance between braced points.

User Note: The stability bracing requirements in Sections 6.2, 6.3, and 6.4 are based on buckling analysis models involving idealizations of common bracing conditions. Computational analysis methods may be used for greater generality, accuracy and efficiency for more complex bracing conditions. The Commentary to 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 to Section E4.

1. Panel Bracing

The panel bracing system shall have the strength and stiffness specified in this section. The connection of the bracing system to the column shall have the strength specified in Section 6.2.2 for a point brace at that location.

User Note: If the stiffness of the connection to the panel bracing system is comparable to the stiffness of the panel bracing system itself, the panel bracing system and its connection to the column function as a panel and point bracing system arranged in series. Such cases may be evaluated using the alternative analysis methods listed in Section 6.1.

In the direction perpendicular to the longitudinal axis of the column, the required shear strength of the bracing system is:

$$V_{br} = 0.0075P$$  \hspace{1cm} (A-6-1)

and, the required shear stiffness of the bracing system is:

$$\beta_{br} = \frac{1}{\phi} \left( \frac{2P}{L_{br}} \right) \text{ (LRFD)}$$  \hspace{1cm} (A-6-2a)
\[ \beta_{br} = \Omega \left( \frac{2P_r}{L_{br}} \right) \]  
(A-6-2b)

where

\[ P_r = \text{required axial strength of the column within the panel under consideration, using LRFD or ASD load combinations, kips (N)} \]

\[ L_{br} = \text{unbraced length within the panel under consideration, in. (mm)} \]

2. **Point Bracing**

In the direction perpendicular to the longitudinal axis of the column, the required strength of end and intermediate point braces is

\[ P_{br} = 0.015P_r \]  
(A-6-3)

and, the required stiffness of the brace is

\[ \beta_{br} = \frac{1}{\phi} \left( \frac{8P_r}{L_{br}} \right) \]  
(LRFD)  
(A-6-4a)

\[ \beta_{br} = \Omega \left( \frac{8P_r}{L_{br}} \right) \]  
(ASD)  
(A-6-4b)

where

\[ P_r = \text{largest of the required axial strengths of the column within the unbraced lengths adjacent to the point brace, using LRFD or ASD load combinations, kips (N)} \]

\[ L_{br} = \text{unbraced length adjacent to the point brace, in. (mm)} \]

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.

6.3. **BEAM BRACING**

Beams shall be restrained against rotation about their longitudinal axis at points of support. When a braced point is assumed in the design between points of support, lateral bracing, torsional bracing, or a combination of the two shall be provided to prevent the relative displacement of the top and bottom flanges (i.e., to prevent twist). In members subject to double curvature bending, the inflection point shall not be considered a braced point unless bracing is provided at that location.
The requirements of this section shall apply to bracing of doubly and singly symmetric I-shaped members subjected to flexure within a plane of symmetry and zero net axial force.

1. Lateral Bracing

Lateral bracing shall be attached at or near the beam compression flange, except as follows:

(a) At the free end of a cantilevered beam, lateral bracing shall be attached at or near the top (tension) flange.
(b) For braced beams subject to double curvature bending, bracing shall be attached at or near both flanges at the braced point nearest the inflection point.

It is permitted to use either panel or point bracing to provide lateral bracing for beams.

1a. Panel Bracing

The panel bracing system shall have the strength and stiffness specified in this section. The connection of the bracing system to the member shall have the strength specified in Section 6.3.1b for a point brace at that location.

User Note: The stiffness contribution of the connection to the panel bracing system should be assessed as provided in the User Note to Section 6.2.1.

The required shear strength of the bracing system is

\[ V_{br} = 0.015 \left( \frac{M_r C_d}{h_o} \right) \]  \hspace{1cm} (A-6-5)

and, the required shear stiffness of the bracing system is

\[ \beta_{br} = \frac{1}{\phi} \left( \frac{4M_r C_d}{L_{br} h_o} \right) \] (LRFD)  \hspace{1cm} (A-6-6a)

\[ \beta_{br} = \Omega \left( \frac{4M_r C_d}{L_{br} h_o} \right) \] (ASD)  \hspace{1cm} (A-6-6b)

\[ \phi = 0.75 \text{ (LRFD)} \hspace{2cm} \Omega = 2.00 \text{ (ASD)} \]

where

\[ C_d = 1.0, \text{ except in the following case:} \]
\[ = 2.0 \text{ for the brace closest to the inflection point in a beam subject to double curvature bending} \]
\[ L_{br} = \text{unbraced length within the panel under consideration, in. (mm)} \]
\[ M_r = \text{required flexural strength of the beam within the panel under consideration, using LRFD or ASD load combinations, kip-in. (N-mm)} \]
\[ h_o = \text{distance between flange centroids, in. (mm)} \]

1b. Point Bracing

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In the direction perpendicular to the longitudinal axis of the beam, the required strength of end and intermediate point braces is

\[ P_{br} = 0.03 \left( \frac{M_r C_d}{h_o} \right) \]  
(A-6-7)

and, the required stiffness of the brace is

\[ \beta_{br} = \frac{1}{\phi} \left( \frac{10M_r C_d}{L_{br} h_o} \right) \]  
(LRFD)  
(A-6-8a)

\[ \beta_{br} = \Omega \left( \frac{10M_r C_d}{L_{br} h_o} \right) \]  
(ASD)  
(A-6-8b)

\[ \phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)} \]

where

\[ L_{br} = \text{ unbraced length adjacent to the point brace, in. (mm)} \]
\[ M_r = \text{ largest of the required flexural strengths of the beam within the unbraced lengths adjacent to the point brace, using LRFD or ASD load combinations, kip-in. (N-mm)} \]

When the unbraced lengths adjacent to a point brace have different 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, \( L_{e,br} \), 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:

\[ M_{br} = 0.03 M_r \]  
(A-6-9)

and, the required flexural stiffness of the brace is:

\[ \beta_{br} = \left( \frac{\beta_T}{\beta_{sec}} \right) \]  
(A-6-10)

where
\[
\beta_T = \frac{1}{\phi} \frac{2.4L}{nEI_{\text{eff}}/C_b} \quad \text{(LRFD)} \quad \text{(A-6-11a)}
\]

\[
\beta_T = \Omega \frac{2.4L}{nEI_{\text{eff}}/C_b} \quad \text{(ASD)} \quad \text{(A-6-11b)}
\]

\[
\beta_{se} = \frac{3.3E}{h_t} \left( \frac{1.5h_t b_t^3}{12} + \frac{t_w b_t^3}{12} \right)
\]

and

\[
\phi = 0.75 \quad \text{(LRFD)}; \quad \Omega = 3.00 \quad \text{(ASD)}
\]

**User Note:** \(\Omega = 1.5^2 / \phi = 3.00\) in Equations A-6-11a or A-6-11b, because the moment term is squared.

\(\beta_{se}\) can be taken equal to infinity, and \(\beta_{se} = \beta_T\), 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

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

\(I_{\text{eff}}\) = effective out-of-plane moment of inertia, in.\(^4\) (mm\(^4\))

\(= I_{yc} + \left( \frac{t}{c} \right) I_{yt}\)

\(I_{yc}\) = 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)

\(M_r\) = 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)

\(= 2\) times 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)

\(\beta_T\) = overall brace system required stiffness, kip-in./rad (N-mm/rad)

\(\beta_{se}\) = web distortional stiffness, including the effect of web transverse stiffeners, if any, kip-in./rad (N-mm/rad)

**User Note:** If \(\beta_{se} < \beta_T\), 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 $I_{out} = \text{out-of-plane moment of inertia, } I_y$, in.$^2$ (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, $M_f$. The required flexural strength, $M_f$, 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:

$$\beta_{sec} = \frac{3.3Ew}{12h_w} \quad \text{(A-6-13)}$$

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, $L_{ue}$ for beam-columns shall be taken as the actual unbraced length; the provisions in Sections 6.2.2 and 6.3.1b, that $L_{ue}$ need not be taken less than the maximum permitted effective length based upon $P_t$ and $M_f$, 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 to 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

7.2. Material Behavior at Elevated Temperatures

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 Equation A-7-1.

For \( f \leq F_y \)

\[
\varepsilon = \frac{f}{E} + 0.002 \left( \frac{f}{F_y} \right)^n \quad (A-7-1a)
\]

For \( F_y < f \leq F_u \)

\[
\varepsilon = 0.002 \left( 1 + \frac{f - F_y}{E_T} \right) + \left( \varepsilon_u - 0.002 \frac{F_u - F_y}{E_T} \right) \left( \frac{f - F_y}{F_u - F_y} \right)^m \quad (A-7-1b)
\]

where

\( E = \) modulus of elasticity of stainless steel

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

\( E_T = \) tangent modulus at the specified minimum yield stress, determined in accordance with Equation A-7-42 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

\( m = 1 + \frac{1}{2} \frac{F_u}{F_y} \quad (A-7-2)\)

\( f = \) engineering stress, ksi (MPa)

\( \varepsilon = \) engineering strain

\( \varepsilon_u = \) strain at the ultimate tensile stress, ultimate strain, which may be approximated in accordance with Equation A-7-32 if not known

\( = 1 - \frac{F_y}{F_u} \leq \varepsilon_f \quad (A-7-32)\)

\( \varepsilon_f = \) specified minimum elongation after fracture 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-7-42 for any stress level $f \leq F_y$.

$$E_T = \frac{E F_y}{F_y + 0.002 n E \left( \frac{f}{F_y} \right)^{n-1}} \quad (A-7-42)$$

3. Secant Modulus, $E_S$

The secant modulus, $E_S$, of austenitic and duplex stainless steel can be determined in accordance with Equation A-7-54 for any stress level $f \leq F_y$.

$$E_S = \frac{E}{1 + 0.002 E \left( \frac{f}{F_y} \right)^n} \quad (A-7-54)$$

**User note:** The secant modulus may be required to check the shear buckling resistance of round HSS, as specified in Section G5 or the deflection of any cross section made of stainless steel, as specified in Section L3.

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 Equation A-7-65. 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 $f(T) \leq F_y(T)$

$$\varepsilon(T) = \frac{f(T)}{E(T)} + 0.002 \left( \frac{f(T)}{F_y(T)} \right)^{n(T)} \quad (A-7-65a)$$

For $F_y(T) < f(T) \leq F_u(T)$

$$\varepsilon(T) = \varepsilon_y(T) + \left( \frac{f(T) - F_y(T)}{E_y(T)} \right) + \left( \frac{E_u(T) - \varepsilon_y(T)}{E_y(T)} \right) \left( \frac{f(T) - F_y(T)}{F_u(T) - F_y(T)} \right)^{n(T)} \quad (A-7-65b)$$

where

$E(T) =$ modulus of elasticity of stainless steel at elevated temperatures, ksi

$E_y(T) =$ tangent modulus at the yield stress at elevated temperatures, ksi

$$E_u(T) = \frac{E(T)}{1 + 0.002 n(T) \left( \frac{E(T)}{F_u(T)} \right)} \quad (A-7-7)$$

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\[ F_y(T) = \text{yield stress at elevated temperatures, ksi (MPa)} \]
\[ = k_y F_y \]
\[ F_u(T) = \text{tensile strength at elevated temperatures, ksi (MPa)} \]
\[ = k_u F_u \]
\[ F_2(T) = \text{stress at 2\% strain at elevated temperatures, ksi (MPa)} \]
\[ = k_2 F_y \]
\[ n(T) = \text{strain hardening coefficient at elevated temperatures} \]
\[ = 7 \text{ for austenitic stainless steel} \]
\[ = 8 \text{ for duplex stainless steel} \]
\[ m(T) = \frac{\ln \left( \frac{0.02 - \varepsilon_y(T)}{\varepsilon_u(T) - \varepsilon_y(T)} \right) - \ln \left( \frac{F_2(T) - F_y(T)}{F_u(T) - F_y(T)} \right)}{n(T) - \ln \left( \frac{F_2(T) - F_y(T)}{F_u(T) - F_y(T)} \right)} \]
\[ f(T) = \text{engineering stress at elevated temperatures, ksi (MPa)} \]
\[ \epsilon(T) = \text{engineering strain at elevated temperatures} \]
\[ \epsilon_y(T) = \text{strain at the yield stress at elevated temperatures} \]
\[ = 0.002 + \frac{F_y(T)}{E(T)} \]  
(A-7-9)
\[ \epsilon_u(T) = \text{ultimate strain at elevated temperatures, which may be approximated as:} \]
\[ = 1 - \frac{F_2(T)}{F_u(T)} \]  
(A-7-10)
\[ k_y, k_u, k_n, \text{ and } k_2 \text{ shall be taken from Appendix 4 Table A-4.2.2, Table A-4.2.3, Table A-4.2.4, or Table A-4.2.5 for the relevant type of stainless steel.} \]