

PART VI
Chapter 10
Steel Structures

10.1 General Provisions for Structural Steel Buildings and Structures

This Section states the scope of the Specification, summarizes referenced Specification, code, and standard documents, and provide requirements for materials and contract documents.

10.1.1 Scope

The specification contained in Chapter 10 Part 6 of this Code sets forth criteria for the design, fabrication, and erection of structural steel buildings and other structures, where other steel-structures are defined as those structures designed, fabricated, and erected in a manner similar to steel-buildings, with building-like vertical and lateral load resisting elements. 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. Alternate methods of analysis and design shall be permitted, provided such alternate methods or criteria are acceptable to the authority having jurisdiction.

10.1.1.1 Low-seismic applications

When the seismic response modification coefficient, R (as specified in Chapter 2 Part 6) is taken equal to or less than 3, the design, fabrication, and erection of structural-steel-framed buildings and other steel-structures shall comply with this specification except that such structures need not to comply with the specifications set forth in Sec 10.20 Seismic Provisions.

10.1.1.2 High-seismic applications

When the seismic response modification coefficient, R (as specified in Chapter 2 Part 6) is taken greater than 3, the design, fabrication and erection of structural-steel-framed buildings and other structures shall comply with the requirements in the Sec 10.20 Seismic Provisions, in addition to the provisions of other sections (whichever applicable) this specification.

10.1.2 Symbols, Glossary and Referenced Specifications, Codes and Standards

10.1.2.1 Symbols

The Section or Table number in the right-hand column refers to where the symbol is first used.

<u>Symbol</u>	<u>Meaning</u>	<u>Section</u>
A	Column cross-sectional area, mm^2	10.10.10.6
A	Total cross-sectional area of member, mm^2	10.5.7.2

<u>Symbol</u>	<u>Meaning</u>	<u>Section</u>
A_{BM}	Cross-sectional area of the base metal, mm ²	10.10.2.4
A_b	Nominal unthreaded body area of bolt or threaded part, mm ²	10.10.3.6
A_b	Cross-sectional area of a horizontal boundary element (HBE), mm ²	10.20.17.2.1
A_{bi}	Cross-sectional area of the overlapping branch, mm ²	10.11.2.3
A_{bj}	Cross-sectional area of the overlapped branch, mm ²	10.11.2.3
A_c	Cross-sectional area of a vertical boundary element (VBE), mm ²	10.20.17.2.1
A_D	Area of an upset rod based on the major thread diameter, mm ²	Table 6.10.10
A_e	Effective net area, mm ²	10.4.2
A_{eff}	Summation of the effective areas of the cross section based on the reduced effective width, b_e , mm ²	10.5.7.2
A_f	Flange area, mm ²	10.20.8
A_{fc}	Area of compression flange	10.7.3.1
A_{fg}	Gross tension flange area, mm ²	10.6.13.1
A_{fn}	Net tension flange area, mm ²	10.6.13.1
A_{ft}	Area of tension flange, mm ²	10.7.3.1
A_g	Gross area of member, mm ²	10.2.3.13
A_g	Gross area of section based on design wall thickness, mm ²	10.7.6
A_g	Chord gross area, mm ²	10.11.2.2
A_g	Gross area, mm ²	10.20.9
A_{gv}	Gross area subject to shear, mm ²	10.10.4.3
A_n	Net area of member, mm ²	10.2.3.13
A_{nt}	Net area subject to tension, mm ²	10.10.4.3
A_{nv}	Net area subject to shear, mm ²	10.10.4.2

<u>Symbol</u>	<u>Meaning</u>	<u>Section</u>
A_{pb}	Projected bearing area, mm ²	10.10.7
A_{sc}	Area of the yielding segment of steel core, mm ²	10.20.16
A_{sf}	Shear area on the failure path, mm ²	10.4.5.1
A_{st}	Stiffener area, mm ²	10.7.3.3
A_{st}	Area of link stiffener, mm ²	10.20.15
A_t	Net tensile area, mm ²	10.17.4
A_w	Web area, the overall depth times the web thickness, dt_w , mm ²	10.7.2.1
A_w	Effective area of the weld, mm ²	10.10.2.4
A_w	Link web area, mm ²	10.20.15
A_{wi}	Effective area of weld throat of any i^{th} weld element, mm ²	10.10.2.4
A_1	Area of steel concentrically bearing on a concrete support, mm ²	10.10.8
A_2	Maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, mm ² .	10.10.8
B	Factor for lateral-torsional buckling in tees and double angles	10.6.9.2
B	Overall width of rectangular HSS member, measured 90° to the plane of connection, mm.	10.11.1.1, Table 6.10.2
B	Overall width of rectangular HSS main member, measured 90° to the plane of the connection, mm.	10.11.2.1, 10.11.3.1
B_b	Overall width of rectangular HSS branch member, measured 90° to the plane of the connection, mm.	10.11.2.1, 10.11.3.1
B_{bi}	Overall branch width of the overlapping branch	10.11.2.3
B_{bj}	Overall branch width of the overlapped branch	10.11.2.3
B_p	Width of plate, measure 90° to the plane of the connection, mm.	10.11.1.1
B_p	Width of plate, transverse to the axis of the main member, mm.	10.11.2.3

<u>Symbol</u>	<u>Meaning</u>	<u>Section</u>
B_1, B_2	Factors used in determining M_u for combined bending and axial forces when first-order analysis is employed	10.3.2.1
C	HSS torsional constant	10.8.3.1
C_a	Ratio of required strength to available strength	Table 6.10.8
C_b	Lateral-torsional buckling modification factor for nonuniform moment diagrams when both ends of the unsupported segment are braced	10.6.1
C_d	Coefficient relating relative brace stiffness and curvature	10.19.3.1, 10.20.9
C_d	Deflection amplification factor	P.2
C_f	Constant based on stress category, given in Table 6.10.14	10.17.3
C_m	Coefficient assuming no lateral translation of the frame	10.3.2.1
C_p	Ponding flexibility coefficient for primary member in a flat roof	10.16.1
C_r	Coefficient for web sidesway buckling	10.10.10.4
C_r	Parameter used for determining the approximate fundamental period	P.2
C_s	Ponding flexibility coefficient for secondary member in a flat roof	10.16.1
C_v	Web shear coefficient	10.7.2.1
C_w	Warping constant, mm ⁶	10.5.4
D	Nominal dead load	10.16.2
D	Dead load due to the weight of the structural elements and permanent features on the building, N.	10.20.9
D	Outside diameter of round HSS, mm.	Table 6.10.8
D	Outside diameter of round HSS member, mm.	Table 6.10.1
D	Outside diameter of round HSS member, mm.	10.11.1.1
D	Outside diameter of round HSS main member, mm.	10.11.2.1
D	Outside diameter of round HSS main member, mm.	10.11.3.1
D	Outside diameter, mm.	10.5.7.2

<u>Symbol</u>	<u>Meaning</u>	<u>Section</u>
D	Chord diameter, mm.	10.11.2.2
D_b	Outside diameter of round HSS branch member, mm.	10.11.2.1
D_b	Outside diameter of round HSS branch member, mm.	10.11.3.1
D_s	Factor used in Eq. 6.10.144, dependent on the type of transverse stiffeners used in a plate girder	10.7.3.3
D_u	In slip-critical connections, a multiplier that reflects the ratio of the mean installed bolt pretension to the specified minimum bolt pretension	10.10.3.8
E	Earthquake load	10.20.4
E	Effect of horizontal and vertical earthquake-induced loads	10.20.9
E	Modulus of elasticity of steel, $E = 200,000$ MPa	10.20.8
E	Eccentricity in a truss connection, positive being away from the branches, mm.	10.11.2.1
E	Modulus of elasticity of steel = 200000 MPa	Table 6.10.1
E_c	Modulus of elasticity of concrete, MPa.	10.18.2.3
E_{cm}	Modulus of elasticity of concrete at elevated temperature, MPa	
EI	Flexural elastic stiffness of the chord members of special segment, (N-mm ²)	10.20.12
E_m	Modulus of elasticity of steel at elevated temperature, MPa.	10.18.2.3
F_a	Available axial stress at the point of consideration, MPa.	10.8.2
F_{BM}	Nominal strength of the base metal per unit area, MPa.	10.10.2.4
F_{BW}	Available flexural stress at the point of consideration about the major axis, MPa.	10.8.2
F_{bz}	Available flexural stress at the point of consideration about the minor axis, MPa.	10.8.2
F_c	Available stress, MPa.	10.11.2.2
F_{cr}	Critical stress, MPa.	10.5.3
F_{cr}	Buckling stress for the section as determined by analysis, MPa.	10.6.12.2

<u>Symbol</u>	<u>Meaning</u>	<u>Section</u>
F_{cry}	Critical stress about the minor axis, MPa.	10.5.4
F_{crz}	Critical torsional buckling stress, MPa.	10.5.4
F_e	Elastic critical buckling stress, MPa.	10.3.1.3
F_{ex}	Elastic flexural buckling stress about the major axis, MPa.	10.5.4
F_{EXX}	Electrode classification number, MPa.	10.10.2.4
F_{ey}	Elastic flexural buckling stress about the minor axis, MPa.	10.5.4
F_{ez}	Elastic torsional buckling stress, MPa.	10.5.4
F_L	A calculated stress used in the calculation of nominal flexural strength, MPa.	Table 6.10.1
F_n	Nominal torsional strength	10.8.3.3
F_n	Nominal tensile stress F_{nt} , or shear stress, F_{nv} , from Table 6.10.10, MPa.	10.10.3.6
F_{nt}	Nominal tensile stress from Table 6.10.10, MPa.	10.10.3.7
F'_{nt}	Nominal tensile stress modified to include the effects of shearing stress, MPa.	10.10.3.7
F_{nv}	Nominal shear stress from Table 6.10.10, MPa.	10.10.3.7
F_{SR}	Design stress range, MPa.	10.17.3
F_{TH}	Threshold fatigue stress range, maximum stress range for indefinite design life from Table 6.10.14, MPa.	10.17.1
F_u	Specified minimum tensile strength of the type of steel being used, MPa.	10.4.2
F_u	Specified minimum tensile strength of the connected material, MPa.	10.10.3.10
F_u	Specified minimum tensile strength of HSS material, MPa.	10.11.1.1
F_u	Specified minimum tensile strength of HSS material, MPa.	10.11.2.1
F_u	Ultimate strength of HSS member, MPa.	10.11.3.1

<u>Symbol</u>	<u>Meaning</u>	<u>Section</u>
F_u	Specified minimum tensile strength, MPa.	10.20.6
F_{um}	Specified minimum tensile strength of the type of steel being used at elevated temperature, MPa.	10.18.2
F_w	Nominal strength of the weld metal per unit area, MPa.	10.10.2.4
F_{wi}	Nominal stress in any i^{th} weld element, MPa.	10.10.2.4
F_{wix}	x component of stress F_{wi} , MPa.	10.10.2.4
F_{wiy}	y component of stress F_{wi} , MPa.	10.10.2.4
F_y	Specified minimum yield stress of the type of steel being used, MPa. As used in this Specification, “yield stress” denotes either the specified minimum yield point (for those steels that have a yield point) or specified yield strength (for those steels that do not have a yield point)	10.20.6, Table 6.10.1
F_y	Specified minimum yield stress of the compression flange, MPa.	10.15.3
F_y	Specified minimum yield stress of the column web, MPa.	10.10.10.6
F_y	Specified minimum yield stress of HSS member material, MPa.	10.11.1.1
F_y	Specified minimum yield stress of HSS main member material, MPa.	10.11.2.1
F_y	Specified minimum yield stress of HSS main member, MPa.	10.11.3.1
F_{yb}	Specified minimum yield stress of HSS branch member material, MPa.	10.11.2.1
F_{yb}	Specified minimum yield stress of HSS branch member, MPa.	10.11.3.1
F_{yb}	F_y of a beam, MPa.	10.20.9
F_{yc}	F_y of a column, MPa.	10.20.9
F_{ybi}	Specified minimum yield stress of the overlapping branch material, MPa.	10.11.2.3
F_{yf}	Specified minimum yield stress of the flange, MPa.	10.10.10.1
F_{ym}	Specified minimum yield stress of the type of steel used at elevated temperature, MPa.	10.18.2
F_{yp}	Specified minimum yield stress of plate, MPa.	10.11.1.1

<u>Symbol</u>	<u>Meaning</u>	<u>Section</u>
F_{ysc}	Specified minimum yield stress of the steel core, or actual yield stress of the steel core as determined from a coupon test, MPa.	10.20.16
F_{yst}	Specified minimum yield stress of the stiffener material, MPa.	10.7.3.3
F_{yw}	Specified minimum yield stress of the web, MPa.	10.10.10.2
G	Shear modulus of elasticity of steel = 77 200 MPa.	10.5.4
G	Gap between toes of branch members in a gapped K-connection, neglecting welds, mm.	10.11.2.1
H	Flexural constant	10.5.4
H	Overall height of rectangular HSS member, measured in the plane of connection, mm.	10.11.1.1
H	Overall height of rectangular HSS main member, measured in plane of connection, mm.	10.11.2.1
H	Overall height of rectangular HSS main member, measured in plane of connection, mm.	10.11.3.1
H	Overall height of rectangular HSS member, measured in the plane of connection, mm.	Table 6.10.2
H	The load length parameter, applicable only to rectangular HSS; the ratio of the length of contact of the branch with the chord in the plane of the connection to the chord width = N/B , where, $N = H_b/\sin\theta$	10.11.2.1
H	Height of story, which may be taken as the distance between the centerline of floor framing at each of the levels above and below, or the distance between the top of floor slabs at each of the levels above and below, mm	10.20.8
H_b	Overall height of rectangular HSS branch member, measured in the plane of the connection, mm.	10.11.3.1
H_b	Overall height of rectangular HSS branch member, measured in the plane of the connection, mm.	10.11.2.1
H_{bi}	Overall depth of the overlapping branch	10.11.2.3
ΣH	Story shear produced by the lateral forces used to compute Δ_H , N.	10.3.2.1
I	Moment of inertia in the place of bending, mm ⁴ .	10.3.2.1

<u>Symbol</u>	<u>Meaning</u>	<u>Section</u>
I	Moment of inertia about the axis of bending, mm ⁴ .	10.14.3
I	Moment of inertia, mm ⁴	10.20.12
I_c	Moment of inertia of a vertical boundary element (VBE) taken perpendicular to the direction of the web plate line, mm ⁴	10.20.17
I_d	Moment of inertia of the steel deck supported on secondary members, mm ⁴	10.16.1
I_p	Moment of inertia of primary members, mm ⁴	10.16.1
I_s	Moment of inertia of secondary members, mm ⁴	10.16.1
I_x, I_y	Moment of inertia about the principal axes, mm ⁴	10.5.4
I_y	Out-of-plane moment of inertia, mm ⁴	10.19.2
I_z	Minor principal axis moment of inertia, mm ⁴	10.6.10.2
I_{yc}	Moment of inertia about y-axis referred to the compression flange, or if reverse curvature bending referred to smaller flange, mm ⁴	10.6.1
J	Torsional constant, mm ⁴	10.5.4
K	Effective length factor determined in accordance with Sec 10.3	10.3.1.2
K	Effective length factor for prismatic member	10.20.13
K_z	Effective length factor for torsional buckling	10.5.4
K_1	Effective length factor in plane of bending, calculated based on the assumption of no lateral translation set equal to 1.0 unless analysis indicates a smaller value to be used.	10.3.2.1
K_2	Effective length factor in the plane of bending, calculated based on a sidesway buckling analysis	10.3.2.1
L	Story height, mm.	10.3.2.1
L	Laterally unbraced length of a member, mm.	10.5.2
L	Length of member between work points at truss chord centerlines, mm.	10.5.5

<u>Symbol</u>	<u>Meaning</u>	<u>Section</u>
L	Length of the member, mm.	10.8.3
L	Actual length of end-loaded weld, mm.	10.10.2.2
L	Nominal occupancy live load	10.18.1.4
L	Span length, mm.	10.19.2
L	Span length of the truss, mm.	10.20.12
L	Distance between VBE centerlines, mm	10.20.17
L_b	Distance between braces, mm.	10.19.2
L_b	Length between points that are either braced against lateral displacement of compression flange or braced against twist of the cross section, mm.	10.6.2, 10.20.13
L_c	Distance between plastic hinge locations, mm	10.20.9
L_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material, mm.	10.10.3.10
L_c	Link length, mm	10.20.15
L_{cf}	Clear distance between VBE flanges, mm	10.20.17
L_e	Total effective weld length of groove and fillet welds to rectangular HSS, mm.	10.11.2.3
L_p	Limiting laterally unbraced length for the limit state of yielding, mm.	10.6.2.2
L_p	Column spacing in direction of girder, m	10.16
L_p	Limiting laterally unbraced length for full plastic flexural strength, uniform moment case, mm.	10.20.12
L_{pd}	Limiting laterally unbraced length for plastic analysis, mm.	10.15.7
L_{pd}	Limiting laterally unbraced length for plastic analysis, mm	10.20.13
L_q	Maximum unbraced length for M_r (the required flexural strength), mm.	10.19.2
L_r	Limiting laterally unbraced length for limit state of inelastic lateral-torsional buckling, mm.	10.6.2.2

<u>Symbol</u>	<u>Meaning</u>	<u>Section</u>
L_s	Column spacing perpendicular to direction of girder, m	10.16.1
L_s	Length of the special segment, mm	10.20.12
L_v	Distance from maximum to zero shear force, mm.	10.7.6
M_A	Absolute value of moment at quarter point of the unbraced segment, N-mm.	10.6.1
M_a	Required flexural strength in chord, using ASD load combinations, N-mm.	10.11.2.2
M_a	Required flexural strength, using ASD load combinations, N-mm.	10.20.9
M_{av}	Additional moment due to shear amplification from the location of plastic hinge to the column centerline based on ASD load combinations, N-mm.	10.20.9
M_B	Absolute value of moment at centerline of the unbraced segment, N-mm.	10.6.1
M_{br}	Required bracing moment, N-mm.	10.19.2
M_C	Absolute value of moment at three-quarter point of unbraced segment, N-mm.	10.6.1
$M_{C(x,y)}$	Available flexural strength determined in accordance with Sec 10.6, N-mm.	10.8.1.1
M_{Cx}	Available flexural-torsional strength for strong axis flexure determined in accordance with Sec 10.6, N-mm.	10.8.1.3
M_e	Elastic lateral-torsional buckling moment, N-mm.	10.6.10.2
M_{lt}	First-order moment under LRFD or ASD load combinations caused by lateral translation of the frame only, N-mm.	10.3.2.1
M_{max}	Absolute value of maximum moment in the unbraced segment, N-mm.	10.6.1
M_n	Nominal flexural strength, N-mm.	10.6.1
M_n	Nominal flexural strength, N-mm.	10.20.11
M_{nc}	Nominal flexural strength of the chord member of special segment, N-mm.	10.20.12
M_{nt}	First-order moment using LRFD or ASD load combinations assuming there is no lateral translation of the frame, N-mm.	10.3.2.1

<u>Symbol</u>	<u>Meaning</u>	<u>Section</u>
M_p	Plastic bending moment, N-mm.	Table 6.10.1
M_p	Nominal plastic flexural strength, N-mm.	Table 6.10.8
M_{pa}	Nominal plastic flexural strength modified by axial load, N-mm.	10.20.15
M_{pb}	Nominal plastic flexural strength of the beam, N-mm.	10.20.9
M_{pc}	Nominal plastic flexural strength of the column, N-mm.	10.20.8
$M_{p,exp}$	Expected plastic moment, N-mm.	10.20.9
M_r	Required second-order flexural strength under LRFD or ASD load combinations, N-mm.	10.3.2.1
M_r	Required flexural strength using LRFD or ASD load combinations, N-mm.	10.8.1
M_r	Required flexural strength in chord, N-mm.	10.11.2.2
M_r	Expected flexural strength, N-mm.	10.20.9
M_{r-ip}	Required in-plane flexural strength in branch, N-mm.	10.11.3.2
M_{r-op}	Required out-of-plane flexural strength in branch, N-mm.	10.11.3.2
M_u	Required flexural strength, using LRFD load combinations, N-mm.	10.20.9
M_u	Required flexural strength in chord, using LRFD load combinations, N-mm.	10.11.2.2
M_{uv}	Additional moment due to shear amplification from the location of plastic hinge to the column centerline based on LRFD load combinations, N-mm.	10.20.9
$M_{u,exp}$	Expected required flexural strength, N-mm.	10.20.15
M_y	Yield moment about the axis of bending, N-mm.	Table 6.10.1
M_1	Smaller moment, calculated from a first-order analysis, at the ends of that portion of the member unbraced in the plane of bending under consideration, N-mm.	10.3.2.1
M_2	Larger moment, calculated from a first-order analysis, at the ends of that portion of the member unbraced in the plane of bending under consideration, N-mm.	10.3.2.1

<u>Symbol</u>	<u>Meaning</u>	<u>Section</u>
N	Length of bearing (not less than k for end beam reactions),mm.	10.10.10.2
N	Bearing length of the load, measured parallel to the axis of the HSS member, (or measured across the width of the HSS in the case of the loaded cap plates), mm.	10.11.1.1
N	Number of stress range fluctuations in design life	10.17.3
N_b	Number of bolts carrying the Applied tension	10.10.3.9
N_i	Additional lateral load	10.3.2.2
N_i	Notional lateral load Applied at level i , N.	10.14.3
N_s	Number of slip planes	10.10.3.8
O_v	Overlap connection coefficient	10.11.2.2
P	Pitch, mm per thread	10.17.4
P_a	Required axial strength of a column using ASD load combinations, N.	10.20.8
P_{ac}	Required compressive strength using ASD load combinations, N.	10.20.9
P_b	Required strength of lateral brace at ends of the link, N.	10.20.15
P_{br}	Required brace strength, N.	10.19.2
P_c	Available axial compressive strength, N.	10.8.1.1
P_c	Available tensile strength, N.	10.8.1.2
P_c	Available axial strength of a column, N.	10.20.9
P_{co}	Available compressive strength out of the plane of bending, N.	10.8.1.3
P_{e1}, P_{e2}	Elastic critical buckling load for braced and unbraced frame, respectively, N.	10.3.2.1
P_{eL}	Euler buckling load, evaluated in the plane of bending, N.	10.14.3

<u>Symbol</u>	<u>Meaning</u>	<u>Section</u>
$P_{l(t,c)}$	First-order axial force using LRFD or ASD load combinations as a result of lateral translation of the frame only (tension or compression, N).	10.3.2.1
$P_{n(t,c)}$	First-order axial force using LRFD or ASD load combinations, assuming there is no lateral translation of the frame (tension or compression, N).	10.3.2.1
P_n	Nominal axial strength, N.	10.4.2
P_n	Nominal axial strength of a column, N.	10.20.8
P_{nc}	Nominal axial compressive strength of diagonal members of the special segment, N.	10.20.12
P_{nt}	Nominal axial tensile strength of diagonal members of special segment, N.	10.20.12
P_{rc}	Required compressive strength using ASD or LRFD load combinations, N.	10.20.9
P_r	Required second-order axial strength using LRFD or ASD load combinations, N.	10.3.2.1
P_r	Required axial compressive strength using LRFD or ASD load combinations, N.	10.3.2.2
P_r	Required tensile strength using LRFD or ASD load combinations, N.	10.8.1.2
P_r	Required strength, N.	10.10.10.6
P_r	Required axial strength in chord, N.	10.11.2.2
P_r	Required axial strength in branch, N.	10.11.3.2
P_r	Required compressive strength, N.	10.20.15
P_u	Required axial strength in compression, N.	10.15.4
P_u	Required axial strength of a column or a link in LRFD load combinations, N.	10.20.8
P_{uc}	Required compressive strength using LRFD load combinations, N.	10.20.9
P_y	Member yield strength, N.	10.3.2.2
P_y	Nominal axial yield strength of a member, equal to $F_y A_g$, N.	Table 6.10.8

<u>Symbol</u>	<u>Meaning</u>	<u>Section</u>
$P_{y_{sc}}$	Axial yield strength of steel core, N.	10.20.16
Q	Full reduction factor for slender compression elements	10.5.7
Q_a	Reduction factor for slender stiffened compression elements	10.5.7.2
Q_b	Maximum unbalanced vertical load effect applied to a beam by the braces, N.	10.20.13
Q_1	Axial forces and moments generated by at least 1.25 times the expected nominal shear strength of the link	10.20.15
Q_f	Chord-stress interaction parameter	10.11.2.2
Q_s	Reduction factor for slender unstiffened compression elements	10.5.7.1
R	Seismic response modification coefficient	10.20.1
R	Seismic response modification coefficient	10.1.1.1
R	Nominal load due to rainwater or snow, exclusive of the ponding contribution, MPa.	10.16.2
R_a	Required strength (ASD)	10.2.3.4
R_{FIL}	Reduction factor for joints using a pair of transverse fillet welds only	10.17.3
R_m	Factor in Eq. 6.10.8 dependent on type of system	10.3.2.1
R_m	Cross-section monosymmetry parameter	10.6.1
R_n	Nominal strength, N.	10.2.3.3
R_n	Nominal strength, N.	10.20.6
R_n	Nominal slip resistance, N.	10.10.3.8
R_{pc}	Web plastification factor	10.6.4.1
R_{PJP}	Reduction factor for reinforced or unreinforced transverse partial-joint-penetration (PJP) groove welds	10.17.3
R_{pt}	Web plastification factor corresponding to the tension flange yielding limit state	10.6.4.4

<u>Symbol</u>	<u>Meaning</u>	<u>Section</u>
R_t	Ratio of the expected tensile strength to the specified minimum tensile strength F_u , as related to overstrength in material yield stress R_y	10.20.6
R_u	Required strength (LRFD)	10.2.3.3
R_u	Required strength	10.20.9
R_v	Panel zone nominal shear strength	10.20.9
R_{wl}	Total nominal strength of longitudinally loaded fillet welds, as determined in accordance with Table 6.10.8	10.10.2.4
R_{wt}	Total nominal strength of transversely loaded fillet welds, as determined in accordance with Table 6.10.8 without the alternate in Sec 10.10.2.4 (a)	10.10.2.4
R_y	Ratio of the expected yield stress to the specified minimum yield stress, F_y	10.20.6
S	Elastic section modulus of round HSS, mm^3	10.6.8.2
S	Lowest elastic section modulus relative to the axis of bending, mm^3	10.6.12
S	Chord elastic section modulus, mm^3	10.11.2.2
S	Spacing of secondary members, m	10.16.1
S_c	Elastic section modulus to toe in compression relative to axis of bending, mm^3 .	10.6.10.3
S_{eff}	Effective section modulus about major axis, mm^3	10.6.7.2
S_{xt}, S_{xc}	Elastic section modulus referred tension and compression flanges, respectively, mm^3	Table 6.10.1
S_x, S_y	Elastic section modulus taken about the principal axes, mm^3	10.6.2.2, F6
S_y	For channels, taken as the minimum section modulus	10.6.6
T	Nominal forces and deformations due to design-basis fire defined in Sec 4.2.1	10.18.1.4
T_a	Tension force due to ASD load combinations, kN.	10.10.3.9

<u>Symbol</u>	<u>Meaning</u>	<u>Section</u>
T_b	Minimum fastener tension given in Table 6.10.9, kN.	10.10.3.8
T_c	Available torsional strength, N-mm.	10.8.3.2
T_n	Nominal torsional strength, N-mm.	10.8.3.1
T_r	Required torsional strength, N-mm.	10.8.3.2
T_u	Tension force due to LRFD load combinations, kN.	10.10.3.9
U	Shear lag factor	10.4.3.3
U	Utilization ratio	10.11.2.2
U_{bs}	Reduction coefficient, used in calculating block shear rupture strength	10.10.4.3
U_p	Stress index	10.16.2
U_s	Stress index	10.16.2
V_a	Required shear strength using ASD load combinations, N.	10.20.9
V_c	Available shear strength, N.	10.7.3.3
V_n	Nominal shear strength, N.	10.7.1
V_n	Nominal shear strength of a member, N.	10.20.15
V_p	Nominal shear strength of an active link, N.	Table 6.10.8
V_{pa}	Nominal shear strength of an active link modified by axial load magnitude, N.	10.20.15
V_{ne}	Expected vertical shear strength of the special segment, N.	10.20.12
V_r	Required shear strength at the location of the stiffener, N.	10.7.3.3
V_r	Required shear strength using LRFD or ASD load combinations, N.	10.8.3.2
V_u	Required shear strength using LRFD load combinations, N.	10.20.10
Y_i	Gravity load from the LRFD load combination or 1.6 times the ASD load combination Applied at level i , N.	10.3.2.2
Y_t	Hole reduction coefficient, N.	10.6.13.1

Symbol	Meaning	Section
Z	Plastic section modulus about the axis of bending, mm^3	10.6.7.1
Z	Plastic section modulus of a member, mm^3 .	10.20.9
Z_b	Branch plastic section modulus about the correct axis of bending, mm^3	10.11.3.3
Z_b	Plastic section modulus of the beam, mm^3 .	10.20.9
Z_c	Plastic section modulus of the column, mm^3 .	10.20.9
Z_x	Plastic section modulus x -axis, mm^3 .	10.20.8
Z_{RBS}	Minimum plastic section modulus at the reduced beam section, mm^3 .	10.20.9
$Z_{x,y}$	Plastic section modulus about the principal axes, mm^3	10.6.2, F6.1
a	Shortest distance from edge of pin hole to edge of member measured parallel to direction of force, mm.	10.4.5.1
a	Distance between connectors in a built-up member, mm.	10.5.6.1
a	Clear distance between transverse stiffeners, mm.	10.6.13.2
a	Half the length of the non-welded root face in the direction of the thickness of the tension-loaded plate, mm.	10.17.3
a	Angle that diagonal members make with the horizontal	10.20.12
a_w	Ratio of two times the web area in compression due to Application of major axis bending moment alone to the area of the compression flange components	10.6.4.2
b	Width of unstiffened compression element; for flanges of I-shaped members and tees, the width b is half the full-flange width, b_f ; for legs of angles and flanges of channels and zees, the width b is the full nominal dimension; for plates, the width b is the distance from free edge to the first row of fasteners or line of welds, or the distance between adjacent lines of fasteners or lines of welds; for rectangular HSS, width b is the clear distance between the webs less the inside corner radius on each side, mm.	10.2.4.1, 10.2.4.2

<u>Symbol</u>	<u>Meaning</u>	<u>Section</u>
b	Full width of longest angle leg, mm.	10.5.7.1
b	Outside width of leg in compression, mm.	10.6.10.3
b	Width of the angle leg resisting the shear force, mm.	10.7.4
b	Width of compression element as defined in Specification Sec 10.2.4.1, mm.	Table 6.10.8
b_{cf}	Width of column flange, mm.	10.10.10.6
b_{cf}	Width of column flange, mm.	10.20.9
b_e	Reduced effective width, mm.	10.5.7.2
b_{eff}	Effective edge distance; the distance from the edge of the hole to the edge of the part measured in the direction normal to the applied force, mm.	10.4.5.1
b_{eoi}	Effective width of the branch face welded to the chord	10.11.2.3
b_{eov}	Effective width of the branch face welded to the overlapped brace.	10.11.2.3
b_f	Flange width, mm.	10.2.4.1
b_f	Flange width, mm.	10.20.9
b_{fc}	Compression flange width, mm.	10.6.4.2
b_{ft}	Width of tension flange, mm.	10.7.3.1
b_l	Longer leg of angle, mm.	10.5.5
b_s	Shorter leg of angle, mm.	10.5.5
b_s	Stiffener width for one-sided stiffeners, mm.	10.19.2
d	Full nominal depth of section, mm.	10.2.4.1
d	Pin diameter, mm.	10.4.5.1
d	Full nominal depth of tee, mm.	10.5.7.1
d	Depth of rectangular bar, mm.	10.6.11.2

Symbol	Meaning	Section
d	Nominal fastener diameter, mm.	10.10.3.3
d	Diameter, mm.	10.10.7
d	Roller diameter, mm.	10.10.7
d	Nominal fastener diameter, mm.	10.20.7
d	Overall beam depth, mm.	10.20.15
d_b	Beam depth, mm.	10.10.10.6
d_b	Nominal diameter (body or shank diameter), mm.	10.17.4
d_c	Column depth, mm.	10.10.10.6
d_c	Overall column depth, mm.	10.20.9
d_z	Overall panel zone depth between continuity plates, mm.	10.20.9
e	Eccentricity in a truss connection, positive being away from the branches, mm.	10.11.2.1
e	EBF link length, mm.	10.20.15
f_a	Required axial stress at point of consideration of LRFD or ASD load combinations, MPa.	10.8.2
$f_{b(w,z)}$	Required flexural stress at the point of consideration (major axis, minor axis) using LRFD or ASD load combinations, MPa.	10.8.2
f'_{cm}	Specified minimum compressive strength of concrete at elevated temperatures, MPa.	10.18.2
f_o	Stress due to D + R (the nominal dead load + the nominal load due to rainwater or snow exclusive of the ponding contribution, MPa.	10.16.2
f_v	Required shear strength per unit area, MPa.	10.10.3.7
g	Transverse center-to-center spacing (gage) between fastener gage lines, mm.	10.2.3.13
g	Gap between toes of branch members in a gapped K-connection, neglecting welds, mm.	10.11.2.1
h	Clear distance between flanges less the fillet or corner radius for rolled shapes; for built-up sections, the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used; for tees, the overall depth; for rectangular HSS, the clear distance between the flanges less the inside corner radius on each side, mm.	10.2.4.2, Table 6.10.8

<u>Symbol</u>	<u>Meaning</u>	<u>Section</u>
h	Distance between centroids of individual components perpendicular to the member axis of buckling,mm.	10.5.6.1
h	Distance between horizontal boundary element centerlines, mm.	10.20.17
h_c	Twice the distance from the centroid to the following: the inside face of the compression flange less the fillet or corner radius, for rolled shapes; the nearest line of fasteners at the compression flange or the inside faces of the compression flange when welds are used, for built-up sections, mm.	10.2.4.2
h_o	Distance between flange centroids, mm.	10.6.2.2
h_o	Distance between flange centroids, mm	10.20.9
h_p	Twice the distance from plastic neutral axis to the nearest line of fasteners at the compression flange or inside face of compression flange when welds are used, mm	10.2.4.2
h_{sc}	Hole factor	10.10.3.8
j	Factor defined by Eq. 6.10.141 for minimum moment of inertia for a transverse stiffener	10.7.2.2
k	Distance from outer face of flange to the web toe of fillet, mm.	10.10.10.2
k	Outside corner radius of HSS, which is permitted to be taken as $1.5t$ if unknown, mm.	10.11.1.3
k_c	Coefficient for slender unstiffened elements, mm.	Table 6.10.1
k_s	Slip-critical combined tension and shear coefficient	10.10.3.9
k_v	Web plate buckling coefficient	10.7.2.1
l	Largest laterally unbraced length along either flange at the point of load, mm.	10.10.10.4
l	Length of bearing,mm.	10.10.7
l	Length of connection in the direction of loading,mm.	Table 6.10.2
l	Unbraced length between stitches of built-up bracing members,mm.	10.20.13
l	Unbraced length of compression or bracing member, mm.	10.20.13

<u>Symbol</u>	<u>Meaning</u>	<u>Section</u>
n	Number of nodal braced points within the span	10.19.2
n	Threads per mm.	10.17.4
p	Ratio of element i deformation to its deformation at maximum stress	10.10.2.4
p	Projected length of the overlapping branch on the chord	10.11.2.2
q	Overlap length measured along the connecting face of the chord beneath the two branches	10.11.2.2
r	Governing radius of gyration, mm.	10.5.2
r	Governing radius of gyration, mm.	10.20.13
r_{crit}	Distance from instantaneous center of rotation to weld element with minimum $\frac{\Delta u}{r_i}$ ratio, mm.	10.10.2.4
r_i	Minimum radius of gyration of individual component in a built-up member, mm.	10.5.6.1
r_{ib}	Radius of gyration of individual component relative to its centroidal axis parallel to member axis of buckling, mm.	10.5.6.1
$\bar{r}_o$	Polar radius of gyration about the shear center, mm.	10.5.4
r_t	Radius of gyration of the flange components in flexural compression plus one-third of the web area in compression due to application of major axis bending moment alone	10.6.4.2
r_{ts}	Effective radius of gyration used in the determination of L_r for the lateral-torsional buckling limit state for major axis bending of doubly symmetric compact I-shaped members and channels	10.6.2.2
r_x	Radius of gyration about geometric axis parallel to connected leg, mm.	10.5.5
r_y	Radius of gyration about y-axis, mm.	10.5.4
r_y	Radius of gyration about y-axis, mm.	10.20.9
r_z	Radius of gyration for the minor principal axis, mm.	10.5.5

<u>Symbol</u>	<u>Meaning</u>	<u>Section</u>
s	Longitudinal center-to-center spacing (pitch) of any two consecutive holes, mm.	10.2.3.13
t	Thickness of element, mm.	10.2.4.2
t	Thickness of element, mm.	Table 6.10.8
t	Wall thickness, mm.	10.5.7.2
t	Angle leg thickness, mm.	10.6.10.2
t	Width of rectangular bar parallel to axis of bending, mm.	10.6.11.2
t	Thickness of connected material, mm.	10.10.3.10
t	Thickness of plate, mm.	10.4.5.1
t	Design wall thickness for HSS equal to 0.93 times the nominal wall thickness for ERW HSS and equal to the nominal wall thickness for SAW HSS, mm.	10.2.3.12
t	Total thickness of fillers, mm.	10.10.5
t	Thickness of connected part, mm.	10.20.7
t	Thickness of column web or doubler plate, mm.	10.20.9
t	Design wall thickness of HSS main member, mm.	10.11.3.1
t	Design wall thickness of HSS main member, mm.	10.11.2.1
t	Design wall thickness of HSS member, mm.	10.11.1.1
t_b	Design wall thickness of HSS branch member, mm.	10.11.2.1
t_b	Design wall thickness of HSS branch member, mm.	10.11.3.1
t_{bf}	Thickness of beam flange, mm.	10.20.9
t_{bj}	Thickness of the overlapped branch, mm.	10.11.2.3
t_{cf}	Thickness of the column flange, mm.	10.10.10.6

<u>Symbol</u>	<u>Meaning</u>	<u>Section</u>
t_{cf}	Thickness of column flange, mm.	10.20.9
t_f	Thickness of the loaded flange, mm.	10.10.10.1
t_f	Thickness of flange, mm.	10.20.17
t_{fc}	Compression flange thickness, mm.	10.6.4.2
t_p	Thickness of plate, mm.	10.11.1.1
t_p	Thickness of the attached transverse plate, mm.	10.11.2.3
t_p	Thickness of tension loaded plate, mm.	10.17.3
t_p	Thickness of panel zone including doubler plates, mm.	10.20.9
t_s	Web stiffener thickness, mm.	10.19.2
t_w	Web thickness, mm.	Table 6.10.1
t_w	Thickness of web, mm.	Table 6.10.8
t_w	Thickness of element, mm.	10.5.7.1
t_w	Column web thickness, mm.	10.10.10.6
t_w	Beam web thickness, mm.	10.19.3
w	Width of cover plate, mm.	10.6.13.3
w	Weld leg size, mm.	10.10.2.2
w	Plate width, mm.	Table 6.10.2
w	Leg size of the reinforcing or contouring fillet, if any, in the direction of the thickness of the tension-loaded plate, mm.	10.17.3
w_z	Width of panel zone between column flanges, mm.	10.20.9
x	Parameter used for determining the approximate fundamental period	Appendix P.2
x_o, y_o	Coordinates of the shear center with respect to the centroid, mm.	10.5.4

<u>Symbol</u>	<u>Meaning</u>	<u>Section</u>
$\bar{x}$	Connection eccentricity,mm.	Table 6.10.2
y	Subscript relating symbol to weak axis	
z	Subscript relating symbol to minor principal axis bending	
Z_b	Minimum plastic section modulus at the reduced beam section, mm ³	10.20.9
α	Factor used in Eq. 6.10.2.2	10.3.2.1
α	Separation ratio for built-up compression members $=h/(2r_{ib})$	10.5.6.1
α	Angle of diagonal members with the horizontal	10.20.12
α	Angle of web yielding in radians, as measured relative to the vertical	10.20.17
β	Reduction factor given by Eq. 6.10.159	10.10.2.2
β	The width ratio; the ratio of branch diameter to chord diameter $= D_b/D$ for round HSS; the ratio of overall branch width to chord width $= B_b/B$ for rectangular HSS	10.11.2.1, 10.11.3.1
β	Compression strength adjustment factor	10.20.16
β_T	Brace stiffness requirement excluding web distortion, N-mm/radian.	10.19.2
β_{br}	Required brace stiffness	10.19.2
β_{eff}	Effective width ratio; the sum of the perimeters of the two branch members in a K-connection divided by eight times the chord width	10.11.2.1
β_{eop}	Effective outside punching parameter	10.11.2.3
β_{sec}	Web distortional stiffness, including the effect of web transverse stiffeners, if any, N-mm/radian.	10.19.2
β_w	Section property for unequal leg angles, positive for short legs in compression and negative for long legs in compression	10.6.10.2
Δ	First-order interstory drift due to the design loads,mm.	10.3.2.2

<u>Symbol</u>	<u>Meaning</u>	<u>Section</u>
Δ	Design story drift	10.20.15
Δ_b	Deformation quantity used to control loading of test specimen (total brace end rotation for the sub-assembly test specimen; total brace axial deformation for the brace test specimen)	Appendix R.2
Δ_{bm}	Value of deformation quantity, Δ_b , corresponding to the design story drift	Appendix R.6
Δ_{by}	Value of deformation quantity, Δ_b , at first significant yield of test specimen	Appendix R.6
Δ_h	First-order interstory drift due to lateral forces, mm.	10.3.2.1
Δ_i	Deformation of weld elements at intermediate stress levels, linearly proportioned to the critical deformation based on distance from instantaneous center of rotation, r_i , mm.	10.10.2.4
Δ_m	Deformation of weld element at maximum stress, mm.	10.10.2.4
Δ_u	Deformation of weld element at ultimate stress (fracture), usually in element furthest from instantaneous center of rotation, mm.	10.10.2.4
δ	Deformation quantity used to control loading of test specimen	Appendix Q.6
δ_y	Value of deformation quantity δ at first significant yield of test specimen	Appendix Q.6
ρ'	Ratio of required axial force P_u to required shear strength V_u of a link	10.20.15
γ	The chord slenderness ratio; the ratio of one-half the diameter to the wall thickness $= D/(2t)$ for round HSS; the ratio of one-half the width to wall thickness $= B/(2t)$ for rectangular HSS	10.11.2.1, 10.11.3.1
ξ	The gap ratio; the ratio of the gap between the branches of a gapped K-connection to the width of the chord $= g/B$ for rectangular HSS	10.11.2.1

<u>Symbol</u>	<u>Meaning</u>	<u>Section</u>
η	The load length parameter, applicable only to rectangular HSS; the ratio of the length of contact of the branch with the chord in the plane of the connection to the chord width $= N/B$, where $N = H_b/\sin \theta$	10.11.2.1, 10.11.3.1
λ	Slenderness parameter	10.6.3
λ_p	Limiting slenderness parameter for compact element	10.2.4
λ_{pf}	Limiting slenderness parameter for compact flange	10.6.3
λ_{pw}	Limiting slenderness parameter for compact web	10.6.4
λ_r	Limiting slenderness parameter for noncompact element	10.2.4
λ_{rf}	Limiting slenderness parameter for noncompact flange	10.6.3
λ_{rw}	Limiting slenderness parameter for noncompact web	10.6.4
λ_p, λ_{ps}	Limiting slenderness parameter for compact element	10.20.8
μ	Mean slip coefficient for class A or B surfaces, as Applicable, or as established by tests	10.10.3.8
ϕ	Resistance factor	10.2.3.3
ϕ	Resistance factor	10.20.6
ϕ_b	Resistance factor for flexure	10.6.1
ϕ_b	Resistance factor for flexure	10.20.8
ϕ_c	Resistance factor for compression	10.5.1
ϕ_c	Resistance factor for compression	10.20.8
ϕ_{sf}	Resistance factor for shear on the failure path	10.4.5.1
ϕ_T	Resistance factor for torsion	10.8.3.1
ϕ_t	Resistance factor for tension	10.4.2
ϕ_v	Resistance factor for shear	10.7.1

<u>Symbol</u>	<u>Meaning</u>	<u>Section</u>
ϕ_v	Resistance factor for shear strength of panel zone of beam-to-column connections	10.20.9
ϕ_v	Resistance factor for shear	10.20.15
ϕ_{total}	Link rotation angle	Appendix Q.2
Ω	Safety factor	10.2.3.4
Ω	Safety factor	10.20.6
Ω_b	Safety factor for flexure	10.6.1
Ω_b	Safety factor for flexure = 1.67	10.20.8
Ω_c	Safety factor for compression	10.5.1
Ω_c	Safety factor for compression = 1.67	10.20.8
Ω_{sf}	Safety factor for shear on the failure path	10.4.5.1
Ω_T	Safety factor for torsion	10.8.3.1
Ω_t	Safety factor for tension	10.4.2
Ω_v	Safety factor for shear	10.7.1
Ω_v	Safety factor for shear strength of panel zone of beam-to-column connections	10.20.9
Ω_o	Horizontal seismic overstrength factor	10.20.4
θ	Angle of loading measured from the weld longitudinal axis, degrees	10.10.2.4
θ	Acute angle between the branch and chord, degrees	10.11.2.1
θ	Acute angle between the branch and chord, degrees	10.11.3.1
θ	Interstory drift angle, radians	Appendix Q.3
ω	Strain hardening adjustment factor	10.20.16
ϵ_{cu}	Strain corresponding to compressive strength of concrete, f'_c	10.18.2
τ_b	Parameter for reduced flexural stiffness using the direct analysis method	10.14.3

<u>Symbol</u>	<u>Meaning</u>	<u>Section</u>
ΣM^*_{pc}	Moment at beam and column centerline determined by projecting the sum of the nominal column plastic moment strength, reduced by the axial stress P_{uc}/A_g , from the top and bottom of the beam moment connection	10.20.9
ΣM^*_{pb}	Moment at the intersection of the beam and column centerlines determined by projecting the beam maximum developed moments from the column face. Maximum developed moments shall be determined from test results	10.20.9

10.1.2.2 Definitions

ACTIVE FIRE PROTECTION	Building materials and systems that are activated by a fire to mitigate adverse effects or to notify people to take some action to mitigate adverse effects.
ADJUSTED BRACE STRENGTH	Strength of a brace in a buckling-restrained braced frame at deformations corresponding to 2.0 times the design story drift.
ALLOWABLE STRENGTH*	Nominal strength divided by the safety factor, R_n/Ω .
ALLOWABLE STRESS	Allowable strength divided by the appropriate section property, such as section modulus or cross-section area.
AMPLIFICATION FACTOR	Multiplier of the results of first-order analysis to reflect second-order effects.
AMPLIFIED SEISMIC LOAD	Horizontal component of earthquake load E multiplied by Ω_o , where E and the horizontal component of E are specified in the Code.
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 Code intended for allowable strength design (allowable stress design).

AUTHORITY HAVING JURISDICTION	Organization, political subdivision, office or individual charged with the responsibility of administering and enforcing the provisions of the Code.
AVAILABLE STRENGTH*	Design strength or allowable strength, as appropriate.
AVAILABLE STRESS*	Design stress or allowable stress, as appropriate.
AVERAGE RIB WIDTH	Average width of the rib of a corrugation in a formed steel deck.
AUTHORITY HAVING JURISDICTION (AHJ)	Organization, political subdivision, office or individual charged with the responsibility of administering and enforcing the provisions of this Code.
BATTEN PLATE	Plate rigidly connected to two parallel components of a built-up column or beam designed to transmit shear between the components.
BEAM	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 bolted connection, limit state of shear forces transmitted by the bolt 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.
BLOCK SHEAR RUPTURE	In a connection, limit state of tension fracture along one path and shear yielding or shear fracture along another path.
BRACED FRAME	An essentially vertical truss system that provides resistance to lateral forces and provides stability for the structural system.
BRANCH FACE	Wall of HSS branch member.
BRANCH MEMBER	For HSS connections, 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	Nominal strength for buckling or instability limit states.
BUCKLING-RESTRAINED BRACED FRAME (BRBF)	Diagonally braced frame satisfying the requirements of Section 16 in which all members of the bracing system are subjected primarily to axial forces and in which the limit state of compression buckling of braces is precluded at forces and deformations corresponding to 2.0 times the design story drift.
BUCKLING-RESTRAINING SYSTEM	System of restraints that limits buckling of the steel core in BRBF. This system includes the casing on the steel core and structural elements adjoining its connections. The buckling-restraining system is intended to permit the transverse expansion and longitudinal contraction of the steel core for deformations corresponding to 2.0 times the design story drift.
BUILT-UP MEMBER, CROSS-SECTION, SECTION, SHAPE	Member, cross-section, section or shape fabricated from structural steel elements that are welded or bolted together.
CAMBER	Curvature fabricated into a beam or truss so as to compensate for deflection induced by loads.
CASING	Element that resists forces transverse to the axis of the brace there by restraining buckling of the core. The casing requires a means of delivering this force to the remainder of the buckling-restraining system. The casing resists little or no force in the axis of the brace.
CHARPY V-NOTCH IMPACT TEST	Standard dynamic test measuring notch toughness of a specimen.
CHORD MEMBER	For HSS, primary member that extends through a truss connection.
CLADDING	Exterior covering of structure.
COLD-FORMED STEEL STRUCTURAL MEMBER	Shape manufactured by press-braking blanks sheared from sheets, cut lengths of coils or plates, or by roll forming cold-or hot-rolled coils or sheets; both forming operations being performed at ambient room temperature, that is, without manifest addition of heat such as would be required for hot forming.
COLUMN	Structural member that has the primary function of resisting axial force.

COLUMN BASE	Assemblage of plates, connectors, bolts, and rods at the base of a column used to transmit forces between the steel superstructure and the foundation.
COMBINED SYSTEM	Structure comprised of two or more lateral load-resisting systems of different type.
COMPACT SECTION	Section capable of developing a fully plastic stress distribution and possessing a rotation capacity of approximately three before the onset of local buckling.
COMPARTMENTATION	The enclosure of a building space with elements that have a specific fire endurance.
COMPLETE-JOINT-PENETRATION GROOVE WELD (CJP)	Groove weld in which weld metal extends through the joint thickness, except as permitted for HSS connections.
COMPOSITE	Condition in which steel and concrete elements and members work as a unit in the distribution of internal forces.
CONCRETE CRUSHING	Limit state of compressive failure in concrete having reached the ultimate strain.
CONCRETE HAUNCH	Section of solid concrete that results from stopping the deck on each side of the girder in a composite floor system constructed using a formed steel deck.
CONCRETE-ENCASED BEAM	Beam totally encased in concrete cast integrally with the slab.
CONNECTION	Combination of structural elements and joints used to transmit forces between two or more members.
CONVECTIVE HEAT TRANSFER	The transfer of thermal energy from a point of higher temperature to a point of lower temperature through the motion of an intervening medium.
CONTINUITY PLATES	Column stiffeners at the top and bottom of the panel zone; also known as transverse stiffeners.
CONTRACTOR	Fabricator or erector, as applicable.
COPE	Cutout made in a structural member to remove a flange and conform to the shape of an intersecting member.
COVER PLATE	Plate welded or bolted to the flange of a member to increase cross-sectional area, section modulus or moment of inertia.

CROSS CONNECTION	HSS connection in which forces in branch members or connecting elements transverse to the main member are primarily equilibrated by forces in other branch members or connecting elements on the opposite side of the main member.
DEMANDCRITICAL WELD	Weld so designated by these Provisions.
DESIGNEARTHQUAKE	The earthquake represented by the design response spectrum as specified in the Code.
DESIGNSTORYDRIFT	Amplified story drift (drift under the design earthquake, including the effects of inelastic action), determined as specified in the Code.
DESIGN-BASIS FIRE	A set of conditions that define the development of a fire and the spread of combustion products throughout a building or portion thereof.
DESIGN LOAD*	Applied load determined in accordance with either LRFD load combinations or ASD load combinations, whichever is applicable.
DESIGN STRENGTH*	Resistance factor multiplied by the nominal strength, ϕR_n .
DESIGN STRESS RANGE	Magnitude of change in stress due to the repeated application and removal of service live loads. For locations subject to stress reversal it is the algebraic difference of the peak stresses.
DESIGN STRESS*	Design strength divided by the appropriate section property, such as section modulus or cross section area.
DESIGN WALL THICKNESS	HSS wall thickness assumed in the determination of section properties.
DIAGONAL BRACING	Inclined structural member carrying primarily axial force in a braced frame.
DIAGONAL STIFFENER	Web stiffener at column panel zone oriented diagonally to the flanges, on one or both sides of the web.
DIAPHRAGM PLATE	Plate possessing in-plane shear stiffness and strength, used to transfer forces to the supporting elements.
DIAPHRAGM	Roof, floor or other membrane or bracing system that transfers in-plane forces to the lateral force resisting system.

DIRECT ANALYSIS METHOD	Design method for stability that captures the effects of residual stresses and initial out-of-plumbness of frames by reducing stiffness and applying notional loads in a second-order analysis.
DIRECT BOND INTERACTION	Mechanism by which force is transferred between steel and concrete in a composite section by bond stress.
DISTORTIONAL FAILURE	Limit state of an HSS truss connection based on distortion of a rectangular HSS chord member into a rhomboidal shape.
DISTORTIONAL STIFFNESS	Out-of-plane flexural stiffness of web.
DOUBLE CURVATURE	Deformed shape of a beam with one or more inflection points within the span.
DOUBLE-CONCENTRATED FORCES	Two equal and opposite forces that form a couple on the same side of the loaded member.
DOUBLER	Plate added to, and parallel with, a beam or column web to increase resistance to concentrated forces.
DRIFT	Lateral deflection of structure.
DUAL SYSTEM	Structural system with the following features (1) an essentially complete space frame that provides support for gravity loads; (2) resistance to lateral load provided by moment frames (SMF, IMF or OMF) that are capable of resisting at least 25 percent of the base shear, and concrete or steel shearwalls, or steel braced frames (EBF, S CBF or OCBF); and (3) each system designed to resist the total lateral load in proportion to its relative rigidity.
DUCTILE LIMIT STATE	Ductile limit states include member and connection yielding, bearing deformation at bolt holes, as well as buckling of members that conform to the width-thickness limitations of Table 6.10.18. Fracture of a member or of a connection, or buckling of a connection element, is not a ductile limit state.
ECCENTRICALLY BRACED FRAME (EBF)	Diagonally braced frame meeting the requirements of Section 15 that has at least one end of each bracing member connected to a beam a short distance from another beam-to-brace connection or a beam-to-column connection.
EFFECTIVE LENGTH FACTOR, K	Ratio between the effective length and the unbraced length of the member.

EFFECTIVE LENGTH	Length of an otherwise identical column with the same strength when analyzed with pinned end conditions.
EFFECTIVE NET AREA	Net area modified to account for the effect of shear lag.
EFFECTIVE SECTION MODULUS	Section modulus reduced to account for buckling of slender compression elements.
EFFECTIVE WIDTH	Reduced width of a plate or slab with an assumed uniform stress distribution which produces the same effect on the behavior of a structural member as the actual plate or slab width with its nonuniform stress distribution.
ELASTIC ANALYSIS	Structural analysis based on the assumption that the structure returns to its original geometry on removal of the load.
ELEVATED TEMPERATURES	Heating conditions experienced by building elements or structures as a result of fire, which are in excess of the anticipated ambient conditions.
ENCASED COMPOSITE COLUMN	Composite column consisting of a structural concrete column and one or more embedded steel shapes.
END PANEL	Web panel with an adjacent panel on one side only.
ENGINEER OF RECORD	Engineer having authority or license from government approved Authority to sign and seal engineering and contract documents.
END RETURN	Length of fillet weld that continues around a corner in the same plane. Engineer of record. Licensed professional responsible for sealing the contract documents. Expansion rocker. Support with curved surface on which a member bears that can tilt to accommodate expansion.
EXPANSION ROLLER	Round steel bar on which a member bears that can roll to accommodate expansion.
EXEMPTED COLUMN	Column not meeting the requirements of Eq. 6.10.300 for SMF.
EXPECTED TENSILE STRENGTH *	Tensile strength of a member, equal to the specified minimum tensile strength, F_u , multiplied by R_t .

EXPECTED YIELD STRENGTH	Yield strength in tension of a member, equal to the expected yield stress multiplied by A_g .
EXPECTED YIELD STRESS	Yield stress of the material, equal to the specified minimum yield stress, F_y , multiplied by R_y .
EYEBAR	Pin-connected tension member of uniform thickness, with forged or thermally cut head of greater width than the body, proportioned to provide approximately equal strength in the head and body.
FACTORED LOAD	Product of a load factor and the nominal load. Fastener. Generic term for bolts, rivets, or other connecting devices.
FATIGUE	Limit state of crack initiation and growth resulting from repeated application of live loads.
FAYING SURFACE	Contact surface of connection elements transmitting a shear force.
FILLED COMPOSITE COLUMN	Composite column consisting of a shell of HSS or pipe filled with structural concrete.
FILLER METAL	Metal or alloy to be added in making a welded joint. Filler. Plate used to build up the thickness of one component. Fillet weld reinforcement. Fillet welds added to groove welds.
FILLET WELD	Weld of generally triangular cross section made between intersecting surfaces of elements.
FIRE	Destructive burning, as manifested by any or all of the following: light, flame, heat, or smoke.
FIRE BARRIER	Element of construction formed of fire-resisting materials and tested in accordance with ASTM Standard E119, or other approved standard fire resistance test, to demonstrate compliance with the Building Code.
FIRE ENDURANCE	A measure of the elapsed time during which a material or assembly continues to exhibit fire resistance.
FIRE RESISTANCE	That property of assemblies that prevents or retards the passage of excessive heat, hot gases or flames under conditions of use and enables them to continue to perform a stipulated function.

FIRE RESISTANCE RATING	The period of time a building element, component or assembly maintains the ability to contain a fire, continues to perform a given structural function, or both, as determined by test or methods based on tests.
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.
FLASHOVER	The rapid transition to a state of total surface involvement in a fire of combustible materials within an enclosure.
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.
FLAT WIDTH	Nominal width of rectangular HSS minus twice the outside corner radius. In absence of knowledge of the corner radius, the flat width may 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.
FORMED SECTION	See cold-formed steel structural member.
FORMED STEEL DECK	In composite construction, steel cold formed into a decking profile used as a permanent concrete form.
FULLY RESTRAINED MOMENT CONNECTION	Connection capable of transferring moment with negligible rotation between connected members.
GAGE	Transverse center-to-center spacing of fasteners.
GAP CONNECTION	HSS truss connection with a gap or space on the chord face between intersecting branch members.

GENERAL COLLAPSE	Limit state of chord plastification of opposing sides of a round HSS chord member at a cross-connection.
GEOMETRIC AXIS	Axis parallel to web, flange or angle leg.
GIRDER FILLER	Narrow piece of sheet steel used as a fill between edge of a deck sheet and flange of a girder in a composite floor system constructed using a formed steel deck.
GIRDER	See Beam.
GIRT	Horizontal structural member that supports wall panels and is primarily subjected to bending under horizontal loads, such as wind load.
GOUGE	Relatively smooth surface groove or cavity resulting from plastic deformation or removal of material.
GRAVITY AXIS	Axis through the center of gravity of a member along its length.
GRAVITY FRAME	Portion of the framing system not included in the lateral load resisting system.
GRAVITY LOAD	Load, such as that produced by dead and live loads, acting in the downward direction.
GRIP (OF BOLT)	Thickness of material through which a bolt passes.
GROOVE WELD	Weld in a groove between connection elements. See also AWS D1.1.
GUSSET PLATE	Plate element connecting truss members or a strut or brace to a beam or column.
HEAT FLUX	Radiant energy per unit surface area.
HEAT RELEASE RATE	The rate at which thermal energy is generated by a burning material.
HORIZONTAL SHEAR	Force at the interface between steel and concrete surfaces in a composite beam.
HSS	Square, rectangular or round hollow structural steel section produced in accordance with a pipe or tubing product specification.
INELASTIC ANALYSIS	Structural analysis that takes into account inelastic material behavior, including plastic analysis.

IN-PLANE INSTABILITY	Limit state of a beam-column bent about its major axis while lateral buckling or lateral-torsional buckling is prevented by lateral bracing.
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.
INTERMEDIATE MOMENT FRAME (IMF)	Moment frame system that meets the requirements of Sec 10.20.10.
INTERSTORY DRIFT ANGLE	Interstory displacement divided by story height, radians.
INVERTED-V-BRACED FRAME	See V-braced frame.
JOINT ECCENTRICITY	For HSS truss connection, perpendicular distance from chord member center of gravity to intersection of branch member work points.
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.
K-AREA	The <i>k-area</i> is the region of the web that extends from the tangent point of the web and the flange-web fillet (AISC “ <i>k</i> ” dimension) a distance of 38 mm in to the web beyond the “ <i>k</i> ” dimension.
K-BRACED FRAME	A bracing configuration in which braces connect to a column at a location with no diaphragm or other out-of-plane support.
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.
KSI	Kip per square inch, a US customary unit of stress.
LOWEST ANTICIPATED SERVICE TEMPERATURE (LAST)	The lowest 1-hour average temperature with a 100-year mean recurrence interval.
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 Code intended for strength design (load and resistance factor design).
LACING	Plate, angle or other steel shape, in a lattice configuration, that connects two steel shapes together.
LAP JOINT	Joint between two overlapping connection elements in parallel planes.
LATERAL BRACING	Diagonal bracing, shear walls or equivalent means for providing in-plane lateral stability.
LATERAL BRACING MEMBER	Member that is designed to inhibit lateral buckling or lateral- torsional buckling of primary framing members.
LATERAL LOAD RESISTING SYSTEM	Structural system designed to resist lateral loads and provide stability for the structure as a whole.
LATERAL LOAD	Load that produced by wind or earthquake effects, acting in a lateral direction.
LATERAL-TORSIONAL BUCKLING	Buckling mode of a flexural member involving deflection normal to the plane of bending occurring simultaneously with twist about 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).
LINK	In EBF, the segment of a beam that is located between the ends of two diagonal braces or between the end of a diagonal brace and a column. The length of the link is defined as the clear distance between the end soft wodiagonal braces or between the diagonal brace and the column face.
LINK INTERMEDIATE WEB STIFFENERS	Vertical web stiffeners placed within the link in EBF.

LINK ROTATION ANGLE	Inelastic angle between the link and the beam outside of the link when the total story drift is equal to the design story drift.
LINK SHEAR DESIGN STRENGTH	Lesser of the available shear strength of the link developed from the moment or shear strength of the link.
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 tensile force.
LOCAL BUCKLING**	Limit state of buckling of a compression element within a cross section.
LOCAL CRIPPLING**	Limit state of local failure of web plate in the immediate vicinity of a concentrated load or reaction.
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 Code intended for strength design (load and resistance factor design).
MAIN MEMBER	For HSS connections, chord member, column or other HSS member to which branch members or other connecting elements are attached.
MEASURED FLEXURAL RESISTANCE	Bending moment measured in a beam at the face of the column, for a beam-to-column test specimen tested in accordance with Appendix S.
MECHANISM	Structural system that includes a sufficient number of real hinges, plastic hinges or both, so as to be able to articulate in one or more rigid body modes.

MILL SCALE	Oxide surface coating on steel formed by the hot rolling process.
MILLED SURFACE	Surface that has been machined flat by a mechanically guided tool to a flat, smooth condition.
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.
NODAL BRACE	Brace that prevents lateral movement or twist independently of other braces at adjacent brace points (see relative brace).
NOMINAL DIMENSION	Designated or theoretical dimension, as in the tables of section properties.
NOMINAL LOAD	Magnitude of the load specified by the Code.
NOMINAL RIB HEIGHT	Height of formed steel deck measured from the underside of the lowest point to the top of the highest point.
NOMINAL STRENGTH*	Strength of a structure or component (without the resistance factor or safety factor applied) to resist load effects, as determined in accordance with this Specification.
NONCOMPACT SECTION	Section that can develop the yield stress in its compression elements before local buckling occurs, but cannot develop a rotation capacity of three.
NONDESTRUCTIVE TESTING	Inspection procedure wherein no material is destroyed and integrity of the material or component is not affected.
NOTCH TOUGHNESS	Energy absorbed at a specified temperature as measured in Charpy V-Notch test.
NOTIONAL LOAD	Virtual load applied in a structural analysis to account for destabilizing effects that are not otherwise accounted for in the design provisions.
OUT-OF-PLANE BUCKLING	Limit state of a beam-column bent about its major axis while lateral buckling or lateral-torsional buckling is not prevented by lateral bracing.

ORDINARY CONCENTRICALLY BRACED FRAME (OCBF)	Diagonally braced frame meeting the requirements of Section 14 in which all members of the bracing system are subjected primarily to axial forces.
ORDINARY MOMENT FRAME (OMF)	Moment frame system that meets the requirements of Sec 10.20.11.
OVERLAP CONNECTION	HSS truss connection in which intersecting branch members overlap.
OVER STRENGTH FACTOR, Ω_o	Factor specified by the Code in order to determine the amplified seismic load, where required by these Provisions.
PANEL ZONE	Web area of beam-to-column connection delineated by extension of beam and column flanges through connection, transmitting moment through a shear panel.
PARTIAL-JOINT- PENETRATION GROOVE WELD (PJP)	Groove weld in which the penetration is intentionally less than the complete thickness of the connected element.
PARTIALLY RESTRAINED MOMENT CONNECTION	Connection capable of transferring moment with rotation between connected members that is not negligible.
PASSIVE FIRE PROTECTION	Building materials and systems whose ability to resist the effects of fire does not rely on any outside activating condition or mechanism.
PERFORMANCE-BASED DESIGN	An engineering approach to structural design that is based on agreed-upon performance goals and objectives, engineering analysis and quantitative assessment of alternatives against those design goals and objectives using accepted engineering tools, methodologies and performance criteria.
PERCENT ELONGATION	Measure of ductility, determined in a tensile test as the maximum elongation of the gage length divided by the original gage length.
PERMANENT LOAD	Load in which variations over time are rare or of small magnitude. All other loads are variable loads.
PIPE	See HSS.
PITCH	Longitudinal center-to-center spacing of fasteners. Center-to-center spacing of bolt threads along axis of bolt.

PLASTIC ANALYSIS	Structural analysis based on the assumption of rigid-plastic behavior, in other words, that equilibrium is satisfied throughout the structure and the stress is at or below the yield stress.
PLASTIC HINGE	Yielded zone that forms in a structural member when the plastic moment is attained. The member is assumed to rotate further as if hinged, except that such rotation is restrained by the plastic moment.
PLASTIC MOMENT	Theoretical resisting moment developed within a fully yielded cross section.
PLASTIC STRESS DISTRIBUTION METHOD	Method for determining the stresses in a composite member assuming that the steel section and the concrete in the cross section are fully plastic.
PLASTIFICATION	In an HSS connection, limit state based on an out-of-plane flexural yield line mechanism in the chord at a branch member connection.
PLATE GIRDER	Built-up beam.
PLUG WELD	Weld made in a circular hole in one element of a joint fusing to another element.
PONDING	Retention of water due solely to the deflection of flat roof framing.
POST-BUCKLING STRENGTH	Load or force that can be carried by an element, member, or frame after initial buckling has occurred.
PREQUALIFIED CONNECTION	Connection that complies with the requirements of Appendix N
PRESCRIPTIVE DESIGN	A design method that documents compliance with general criteria established in a Code.
PRETENSIONED JOINT	Joint with high-strength bolts tightened to the specified minimum pretension.
PROPERLY DEVELOPED	Reinforcing bars detailed to yield in a ductile manner before crushing of the concrete occurs. Bars meeting the provisions of Chapter 6 (of Part 6) insofar as development length, spacing and cover shall be deemed to be properly developed.
PROTECTED ZONE	Area of members in which limitations apply to fabrication and attachments.
PROTOTYPE	The connection or brace design that is to be used in the building (SMF, IMF, EBF, and BRBF).

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	Component of branch member force perpendicular to a chord.
PURLIN	Horizontal structural member that supports roof deck and is primarily subjected to bending under vertical loads such as snow, wind or dead loads.
P- δ EFFECT	Effect of loads acting on the deflected shape of a member between joints or nodes.
P- Δ EFFECT	Effect of loads acting on the displaced location of joints or nodes in a structure. In tiered building structures, this is the effect of loads acting on the laterally displaced location of floors and roofs.
QUALITY ASSURANCE	System of shop and field activities and controls implemented by the owner or his/her designated representative to provide confidence to the owner and the building authority that quality requirements are implemented.
QUALITY ASSURANCE PLAN	Written description of qualifications, procedures, quality inspections, resources, and records to be used to provide assurance that the structure complies with the engineer's quality requirements, specifications and contract documents.
QUALITY CONTROL	System of shop and field controls implemented by the fabricator and erector to ensure that contract and company fabrication and erection requirements are met.
RATIONAL ENGINEERING ANALYSIS	Analysis based on theory that is appropriate for the situation, relevant test data if available, and sound engineering judgment.
REDUCED BEAM SECTION	Reduction in cross section over a discrete length that promotes a zone of inelasticity in the member.
REENTRANT	In a cope or weld access hole, a cut at an abrupt change in direction in which the exposed surface is concave.
RELATIVE BRACE	Brace that controls relative movement of two adjacent brace points along length of a beam or column or relative lateral displacement of two stories in a frame.

REQUIRED STRENGTH*	Forces, stresses and deformations acting on the structural component, determined by either structural analysis, for the LRFD or ASD load combinations, as appropriate, or as specified in this Chapter.
RESISTANCE FACTOR, F	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 substantial thermal expansion throughout the range of anticipated elevated temperatures.
REVERSE CURVATURE	See double curvature.
ROOT OF JOINT	Portion of a joint to be welded where the members are closest to each other.
ROTATION CAPACITY	Incremental angular rotation that a given shape can accept prior to excessive load shedding, defined as the ratio of the inelastic rotation attained to the idealized elastic rotation at first yield.
RUPTURE STRENGTH	In a connection, strength limited by tension or shear rupture.
SAFETY FACTOR, Ω	Factor that accounts for deviations of the actual strength from the nominal strength, deviations of the actual load from the nominal load, uncertainties in the analysis that transforms the load into a load effect, and for manner and consequences of failure.
SECOND-ORDER ANALYSIS	Structural analysis in which equilibrium conditions are formulated on the deformed structure; second-order effects (both $P-\Delta$ and $P-\delta$, unless specified otherwise) are included.
SECOND-ORDER EFFECT	Effect of loads acting on the deformed configuration of a structure; includes $P-\Delta$ effect and $P-\delta$ effect.
SEISMIC DESIGN CATEGORY	Classification assigned to a building by the Code based upon its seismic use group and the design spectral response acceleration coefficients.
SEISMIC LOAD RESISTING SYSTEM (SLRS)	Assembly of structural elements in the building that resists seismic loads, including struts, collectors, chords, diaphragms and trusses.

SEISMIC RESPONSE MODIFICATION COEFFICIENT	Factor that reduces seismic load effects to strength level as specified by the Code.
SEISMIC USE GROUP	Classification assigned to a structure based on its use as specified by the Code.
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 or the comfort of its occupants or function of machinery, under normal 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 CONNECTOR	Headed stud, channel, plate or other shape welded to a steel member and embedded in concrete of a composite member to transmit shear forces at the interface between the two materials.
SHEAR CONNECTOR STRENGTH	Limit state of reaching the strength of a shear connector, as governed by the connector bearing against the concrete in the slab or by the tensile strength of the connector.
SHEAR RUPTURE	Limit state of rupture (fracture) due to shear.
SHEAR WALL	Wall that provides resistance to lateral loads in the plane of the wall and provides stability for the structural system.
SHEAR YIELDING	Yielding that occurs due to shear.
SHEAR YIELDING (PUNCHING)	In an HSS connection, limit state based on out-of-plane shear strength of the chord wall to which branch members are attached.
SHEET STEEL	In a composite floor system, steel used for closure plates or miscellaneous trimming in a formed steel deck.
SHIM	Thin layer of material used to fill a space between faying or bearing surfaces.
SIDESWAY BUCKLING	Limit state of lateral buckling of the tension flange opposite the location of a concentrated compression force.

SIDEWALL CRIPPLING	Limit state of web crippling of the sidewalls of a chord member at a HSS truss connection.
SIDEWALL CRUSHING	Limit state based on bearing strength of chord member sidewall in HSS truss connection.
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 forces 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 Sec 10.10.
SPECIAL CONCENTRICALLY BRACED FRAME (SCBF)	Diagonally braced frame meeting the requirements of Sec 10.20.13 in which all members of the bracing system are subjected primarily to axial forces.
SPECIAL MOMENT FRAME (SMF)	Moment frame system that meets the requirements of Sec 10.20.9.
SPECIAL PLATE SHEAR WALL (SPSW)	Plate shear wall system that meets the requirements of Sec 10.20.17.
SPECIAL TRUSS MOMENT FRAME (STMF)	Truss moment frame system that meets the requirements of Sec 10.20.12.
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 reached 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.
STIFFENED ELEMENT	Flat compression element with adjoining out-of-plane elements along both edges parallel to the direction of loading.
STIFFENER	Structural element, usually 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).
STRAIN COMPATIBILITY METHOD	Method for determining the stresses in a composite member considering the stress-strain relationships of each material and its location with respect to the neutral axis of the cross section.
STRENGTH LIMIT STATE	Limiting condition affecting the safety of the structure, in which the ultimate load-carrying capacity is reached.
STRESS	Force per unit area caused by axial force, moment, shear or torsion.
STRESS CONCENTRATION	Localized stress considerably higher than average (even in uniformly loaded cross sections of uniform thickness) 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 STEEL	Steel elements as defined in Sec 10.1.3.
STRUCTURAL SYSTEM	An assemblage of load-carrying components that are joined together to provide interaction or interdependence.
STATIC YIELD STRENGTH	Strength of a structural member or connection determined on the basis of testing conducted under slow monotonic loading until failure.

STEEL CORE. AXIAL-FORCE-RESISTING ELEMENT OF BRACES IN BRBF	The steel core contains a yielding segment and connections to transfer its axial force to adjoining elements; it may also contain projections beyond the casing and transition segments between the projections and yielding segment.
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 RUPTURE	Limit state of rupture (fracture) due to tension.
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.
TENSILE YIELDING	Yielding that occurs due to tension.
TENSION AND SHEAR RUPTURE	In a bolt, limit state of rupture (fracture) 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.
TESTED CONNECTION	Connection that complies with the requirements of Appendix O.
THERMALLY CUT	Cut with gas, plasma or laser.
TIE PLATE	Plate element used to join two parallel components of a built-up column, girder or strut rigidly connected to the parallel components and designed to transmit shear between them.
TOE OF FILLET	Junction of a fillet weld face and base metal. Tangent point of a rolled section fillet.
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.
TORSIONAL YIELDING	Yielding that occurs due to torsion.
TRANSVERSE REINFORCEMENT	Steel reinforcement in the form of closed ties or welded wire fabric providing confinement for the concrete surrounding the steel shape core in an encased concrete composite column.

TRANSVERSE STIFFENER	Web stiffener oriented perpendicular to the flanges, attached to the web.
TUBING	See HSS.
TURN-OF-NUT METHOD	Procedure whereby the specified pretension in high-strength bolts is controlled by rotating the fastener component a predetermined amount after the bolt has been snug tightened.
UNBRACED LENGTH	Distance between braced points of a member, measured between the centers of gravity of the bracing members.
UNEVEN LOAD DISTRIBUTION	In an HSS connection, condition in which the load is not distributed through the cross section of connected elements in a manner that can be readily determined.
UNFRAMED END	The end of a member not restrained against rotation by stiffeners or connection elements.
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.
UNSTIFFENED ELEMENT	Flat compression element with an adjoining out-of-plane element along one edge parallel to the direction of loading.
V-BRACED FRAME	Concentrically braced frame (SCBF, OCBF or BRBF) in which a pair of diagonal braces located either above or below a beam is connected to a single point within the clear beam span. Where the diagonal braces are below the beam, the system is also referred to as an inverted-V-braced frame.
VARIABLE LOAD	Load not classified as permanent load.
VERTICAL BRACING SYSTEM	System of shear walls, braced frames or both, extending through one or more floors of a building.
WEAK AXIS	Minor principal centroidal axis of a cross section.
WEATHERING STEEL	High-strength, low-alloy steel that, with suitable precautions, can be used in normal atmospheric exposures (not marine) without protective paint coating.
WEB BUCKLING	Limit state of lateral instability of a web.

WEB COMPRESSION BUCKLING	Limit state of out-of-plane compression buckling of the web due to a concentrated compression force.
WEB SIDESWAY BUCKLING	Limit state of lateral buckling of the tension flange opposite the location of a concentrated compression force.
WELD METAL	Portion of a fusion weld that has been completely melted during welding. Weld metal has elements of filler metal and base metal melted in the weld thermal cycle.
WELD ROOT	See root of joint.
X-BRACED FRAME	Concentrically braced frame (OCBF or SCBF) in which a pair of diagonal braces crosses near the mid-length of the braces.
Y-BRACED FRAME	Eccentrically braced frame (EBF) in which the stem of the Y is Link the EBF system.
Y-CONNECTION	HSS connection in which the branch member or connecting element is not perpendicular to the main member and in which forces transverse to the main member are primarily equilibrated by shear in the main member.
YIELD MOMENT	In a member subjected to bending, the moment at which the extreme outer fiber first attains the yield stress.
YIELD POINT	First stress in a material at which an increase in strain occurs without an increase in stress as defined by ASTM.
YIELD STRENGTH	Stress at which a material exhibits a specified limiting deviation from the proportionality of stress to strain as defined by ASTM.
YIELD STRESS	Generic term to denote either yield point or yield strength, as appropriate for the material.
YIELDING	Limit state of inelastic deformation that occurs after the yield stress is reached.
YIELDING (PLASTIC MOMENT)	Yielding throughout the cross section of a member as the bending moment reaches the plastic moment.
YIELDING (YIELD MOMENT)	Yielding at the extreme fiber on the cross section of a member when the bending moment reaches the yield moment.

Notes:

- (1) Terms designated with * are usually qualified by the type of load effect, for example, nominal tensile strength, available compressive strength, design flexural strength.
- (2) Terms designated with ** are usually qualified by the type of component, for example, web local buckling, flange local bending.

10.1.2.3 Referenced specifications, codes and standards

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

ACI International (ACI)

ACI 318	Building Code Requirements for Structural Concrete and Commentary
ACI 318M	Metric Building Code Requirements for Structural Concrete and Commentary

American Institute of Steel Construction, Inc. (AISC)

AISC 303	Code of Standard Practice for Steel Buildings and Bridges
ANSI/AISC 341	Seismic Provisions for Structural Steel Buildings
ANSI/AISC N690	Specification for the Design, Fabrication and Erection of Steel Safety-Related Structures for Nuclear Facilities, including Supplement No. 2
ANSI/AISC N690L	Load and Resistance Factor Design Specification for Steel Safety-Related Structures for Nuclear Facilities

American Society of Civil Engineers (ASCE)

SEI/ASCE 7	Minimum Design Loads for Buildings and Other Structures
ASCE/SFPE 29	Standard Calculation Methods for Structural Fire Protection

American Society of Mechanical Engineers (ASME)

ASME B18.2.6	Fasteners for Use in Structural Applications
ASME B46.1	Surface Texture, Surface Roughness, Waviness, and Lay

American Iron and Steel Institute (AISI)

North American Specification for the Design of Cold Formed Steel Structural Members (AISI/COS/NASPEC 2001).

Code of Standard Practice for Cold-Formed Steel Structural Framing

ASTM International (ASTM)

ASTM A6/A6M-04a	Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling
ASTM A36/A36M	Standard Specification for Carbon Structural Steel
ASTM A53/A53M	Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless
ASTM A193/A193M	Standard Specification for Alloy-Steel and Stainless Steel Bolting Materials for High-Temperature Service
ASTM A194/A194M	Standard Specification for Carbon and Alloy Steel Nuts for Bolts for High Pressure or High-Temperature Service, or Both
ASTM A216/A216M	Standard Specification for Steel Castings, Carbon, Suitable for Fusion Welding, for High Temperature Service
ASTM A242/A242M	Standard Specification for High-Strength Low-Alloy Structural Steel
ASTM A283/A283M	Standard Specification for Low and Intermediate Tensile Strength Carbon Steel Plates
ASTM A307	Standard Specification for Carbon Steel Bolts and Studs, 60,000 psi Tensile Strength
ASTM A325	Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength
ASTM A325M	Standard Specification for High-Strength Bolts for Structural Steel Joints (Metric)
ASTM A354	Standard Specification for Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners
ASTM A370	Standard Test Methods and Definitions for Mechanical Testing of Steel Products
ASTM A449	Standard Specification for Quenched and Tempered Steel Bolts and Studs
ASTM A490	Standard Specification for Heat-Treated Steel Structural Bolts, 150 ksi Minimum Tensile Strength
ASTM A490M	Standard Specification for High-Strength Steel Bolts, Classes 10.9 and 10.9.3, for Structural Steel Joints (Metric)
ASTM A500	Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes
ASTM A501	Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing
ASTM A502	Standard Specification for Steel Structural Rivets

ASTM A514/A514M	Standard Specification for High-Yield Strength, Quenched and Tempered Alloy Steel Plate, Suitable for Welding
ASTM A529/A529M	Standard Specification for High-Strength Carbon-Manganese Steel of Structural Quality
ASTM A563	Standard Specification for Carbon and Alloy Steel Nuts
ASTM A563M	Standard Specification for Carbon and Alloy Steel Nuts [Metric]
ASTM A568/A568M	Standard Specification for Steel, Sheet, Carbon, and High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, General Requirements for
ASTM A572/A572M	Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel.
ASTM A588/A588M	Standard Specification for High-Strength Low-Alloy Structural Steel with 345 MPa Minimum Yield Point to 100 mm Thick
ASTM A606	Standard Specification for Steel, Sheet and Strip, High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance
ASTM A618/A618M	Standard Specification for Hot-Formed Welded and Seamless High-Strength Low-Alloy Structural Tubing
ASTM A673/A673M	Standard Specification for Sampling Procedure for Impact Testing of Structural Steel
ASTM A668/A668M	Standard Specification for Steel Forgings, Carbon and Alloy, for General Industrial Use
ASTM A709/A709M	Standard Specification for Carbon and High-Strength Low-Alloy Structural Steel Shapes, Plates, and Bars and Quenched-and-Tempered Alloy Structural Steel Plates for Bridges
ASTM A751	Standard Test Methods, Practices, and Terminology for Chemical Analysis of Steel Products
ASTM A847	Standard Specification for Cold-Formed Welded and Seamless High-Strength, Low-Alloy Structural Tubing with Improved Atmospheric Corrosion Resistance
ASTM A852/A852M	Standard Specification for Quenched and Tempered Low-Alloy Structural Steel Plate with 485 MPa Minimum Yield Strength to 100 mm Thick
ASTM A913/A913M	Standard Specification for High-Strength Low-Alloy Steel Shapes of Structural Quality, Produced by Quenching and Self-Tempering Process (QST)
ASTM A992/A992M	Standard Specification for Structural Steel Shapes

ASTM A1011/A1011M	Standard Specification for Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with Improved Formability
ASTM C33	Standard Specification for Concrete Aggregates
ASTM C330	Standard Specification for Lightweight Aggregates for Structural Concrete
ASTM E119	Standard Test Methods for Fire Tests of Building Construction and Materials
ASTM E709	Standard Guide for Magnetic Particle Examination
ASTM F436	Standard Specification for Hardened Steel Washers
ASTM F959	Standard Specification for Compressible-Washer-Type Direct Tension Indicators for Use with Structural Fasteners
ASTM F1554	Standard Specification for Anchor Bolts, Steel, 36, 55, and 105 ksi Yield Strength
ASTM F1852	Standard Specification for “Twist-Off” Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength

American Welding Society (AWS)

AWS D1.1/D1.1M	Structural Welding Code–Steel
AWS A5.1	Specification for Carbon Steel Electrodes for Shielded Metal Arc Welding
AWS A5.5	Specification for Low-Alloy Steel Electrodes for Shielded Metal Arc Welding
AWS A5.17/A5.17M	Specification for Carbon Steel Electrodes and Fluxes for Submerged Arc Welding
AWS A5.18	Specification for Carbon Steel Electrodes and Rods for Gas Shielded Arc Welding
AWS A5.20	Specification for Carbon Steel Electrodes for Flux Cored Arc Welding
AWS A5.23/A5.23M	Specification for Low-Alloy Steel Electrodes and Fluxes for Submerged Arc Welding
AWS A5.25/A5.25M	Specification for Carbon and Low-Alloy Steel Electrodes and Fluxes for Electroslag Welding
AWS A5.26/A5.26M	Specification for Carbon and Low-Alloy Steel Electrodes for Electrode Gas Welding
AWS A5.28	Specification for Low-Alloy Steel Electrodes and Rods for Gas Shielded Arc Welding
AWS A5.29	Specification for Low-Alloy Steel Electrodes for Flux Cored Arc Welding

Research Council on Structural Connections (RCSC)

Specification for Structural Joints Using ASTM A325 or A490 Bolts, 2004

Bangladesh Standards and Testing Institute (BSTI)

BDS 1031	Mild Steel (MS) Pipe and Galvanized Iron (GI) Pipe
BDS ISO 6935Parts 1 and 2	Steel for Reinforcement of Concrete.
BDS 1122	GP Sheet with Corrugation
BDS EN 197-1Cement Part 1	Composition. Specifications and Conformity Criteria for Common Cement
BDS 208	Common Building Clay Bricks
BDS ISO 13006	Ceramic Tiles
BDS 1825	Aluminium and Aluminium Alloys - Extruded Rod/Bar Tube and Profiles - Tolerances on Dimensions and Form.

10.1.3 Material**10.1.3.1 Structural steel materials**

- (a) Regular Structural Steel: Material test reports from an acceptable testing laboratory shall constitute sufficient evidence of conformity with one of the above listed ASTM standards. For hot-rolled structural shapes, plates, and bars, such tests shall be made in accordance with ASTM A6/A6M; for sheets, such tests shall be made in accordance with ASTM A568/A568M; for tubing and pipe, such tests shall be made in accordance with the requirements of the applicable ASTM standards listed above for those product forms. If requested, the fabricator shall provide an affidavit stating that the structural steel furnished meets the requirements of the grade specified.

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

(i) Hot-rolled structural shapes

ASTM A36/A36M, ASTM A529/A529M, ASTM A572/A572M, ASTM A588/A588M, ASTM A709/A709M, ASTM A913/A913M, ASTM A992/ A992M

(ii) Structural tubing

ASTM A500, ASTM A501, ASTM A618, ASTM A847, BDS 1031

(iii) Pipe

ASTM A53/A53M, Gr. B, BDS 1031

(iv) Plates

ASTM A36/A36M, ASTM A242/A242M, ASTM A283/A283M, ASTM A514/A514M, ASTM A529/A529M, ASTM A572/A572M, ASTM A588/A588M, ASTM A709/A709M, ASTM A852/A852M, ASTM A1011/A1011M, BDS 1122

(v) Bars

ASTM A36/A36M, ASTM A529/A529M, ASTM A572/A572M, ASTM A709/A709M, BDS ISO 6935-1, BDS ISO 6935-2

(vi) Sheets

ASTM A606, A1011/A1011M SS, HSLAS, AND HSLAS-F, BDS 1122

- (b) Unidentified Steel: Unidentified steel free of injurious defects is permitted to be used for unimportant members or details, where the precise physical properties and weldability of the steel would not affect the strength of the structure.
- (c) Rolled Heavy Shapes: ASTM A6/A6M hot-rolled shapes with a flange thickness exceeding 50 mm, used as members subject to primary (computed) tensile forces due to tension or flexure and spliced using complete-joint-penetration groove welds that fuse through the thickness of the member, shall be specified as follows. The contract documents shall require that such shapes be supplied with Charpy V-Notch (CVN) impact test results in accordance with ASTM A6/A6M, Supplementary Requirement S30, Charpy V-Notch Impact Test for Structural Shapes – Alternate Core Location. The impact test shall meet a minimum average value of 27 Joules absorbed energy at +21⁰ C.

The above requirements do not apply if the splices and connections are made by bolting. The above requirements do not apply to hot-rolled shapes with a flange thickness exceeding 50 mm that have shapes with flange or web elements less than 50 mm thick welded with complete-joint-penetration groove welds to the face of the shapes with thicker elements.

- (d) Built-Up Heavy Shapes: Built-up cross-sections consisting of plates with a thickness exceeding 50 mm, 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 contract documents shall require that the steel be supplied with Charpy V-Notch impact test results in accordance with ASTM A6/A6M, Supplementary Requirement S5, Charpy V-Notch Impact Test. The impact test shall be conducted in accordance with ASTM A673/A673M, Frequency P, and shall meet a minimum average value of 27 Joules absorbed energy at +21⁰ C.

The above requirements also apply to built-up cross-sections consisting of plates exceeding 50 mm that are welded with complete-joint-penetration groove welds to the face of other sections.

- (e) Cold Form Sections: Specifications for cold form shapes regarding their use as structural members is not covered in Sec 10. For such type of structural steel, AISI standard (AISI/COS/NASPEC 2001) or equivalent may be followed.

10.1.3.2 Steel castings and forgings

Cast steel shall conform to ASTM A216/A216M, Gr. WCB with Supplementary Requirement S11. Steel forgings shall conform to ASTM A668/A668M. Test reports produced in accordance with the above reference standards shall constitute sufficient evidence of conformity with such standards.

10.1.3.3 Bolts, washers and nuts

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

- (1) Bolts: ASTM A307, ASTM A325, ASTM A325M, ASTM A449, ASTM A490, ASTM A490M, ASTM F1852
- (2) Nuts: ASTM A194/A194M, ASTM A563, ASTM A563M
- (3) Washers: ASTM F436, ASTM F436M
- (4) Compressible-Washer-Type Direct Tension Indicators: ASTM F959, ASTM F959M

10.1.3.4 Anchor rods and threaded rods

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

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

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

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.

10.1.3.5 Filler metal and flux for welding

Filler metals and fluxes shall conform to one of the following specifications of the American Welding Society: AWS A5.1, AWS A5.5, AWS A5.17/A5.17M, AWS A5.18, AWS A5.20, AWS A5.23/A5.23M, AWS A5.25/A5.25M, AWS A5.26/A5.26M, AWS A5.28, AWS A5.29, AWS A5.32/A5.32M

10.1.3.6 Stud shear connectors

Steel stud shear connectors shall conform to the requirements of Structural Welding Code—Steel, AWS D1.1.

10.1.4 Structural Design Drawings and Specifications

The design drawings and specifications shall meet the requirements specified in this specification (Sections 10.1 to 10.20) and shall be prepared and presented in an internationally approved standard in accordance with the provisions of Sec 10.13, except for deviations specifically identified in the design drawings and/or specifications and approved by an appropriate authority.

10.2 General Design Requirements

The general requirements for the analysis and design of steel buildings and structures that are applicable to all Sections of Chapter 10 Part 6 are given in this Section.

10.2.1 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. Unless restricted by the Code, lateral load resistance and stability may be provided by any combination of members and connections.

10.2.2 Loads and Load Combinations

The loads and load combinations shall be as stipulated in Chapter 2 Part 6 of this Code. For design purposes, the nominal loads shall be taken as the loads stipulated in the Chapter 2

10.2.3 Design Basis

Designs shall be made according to the provisions for Load and Resistance Factor Design (LRFD) or to the provisions for Allowable Strength Design (ASD).

10.2.3.1 Required strength

The required strength of structural members and connections shall be determined by structural analysis for the appropriate load combinations as stipulated in Chapter 2 of Part 6. Design by elastic, inelastic or plastic analysis is permitted. Provisions for inelastic and plastic analysis are as stipulated in Sec 10.15, Inelastic Analysis and Design. The provisions for moment redistribution in continuous beams in Sec 10.15.3 are permitted for elastic analysis only.

10.2.3.2 Limit states

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

10.2.3.3 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 as specified in Chapter 2 of Part 6. All provisions of this Specification, except for those in Sec 10.2.3.4, shall apply.

Design shall be performed in accordance with Eq.6.10.1:

$$R_u \leq \phi R_n \quad (6.10.1)$$

Where,

R_u = required strength (LRFD)

R_n = nominal strength, specified in Sections 10.2 to 10.20

ϕ = resistance factor, specified in Sections 10.2 to 10.20

ϕR_n = design strength

10.2.3.4 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 as specified in Chapter 2 of Part 6. All provisions of this Specification, except those of Sec 10.2.3.3, shall apply.

Design shall be performed in accordance with Eq.6.10.2:

$$R_a \leq R_n / \Omega \quad (6.10.2)$$

Where,

R_a = required strength (ASD)

R_n = nominal strength, specified in Sections 10.2 to 10.20

Ω = safety factor, specified in Sections 10.2 to 10.20

R_n/Ω = allowable strength

10.2.3.5 Design for stability

Stability of the structure and its elements shall be determined in accordance with Sec 10.3.

10.2.3.6 Design for connection

Connection elements shall be designed in accordance with the provisions of Sections 10.10 and 10.11. The forces and deformations used in design shall be consistent with the intended performance of the connection and the assumptions used in the structural analysis.

10.2.3.6.1 Simple connection

A simple connection transmits a negligible moment across the connection. 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. Inelastic rotation of the connection is permitted.

10.2.3.6.2 Moment connection

A moment connection transmits moment across the connection. Two types of moment connections, FR and PR, are permitted, as specified below.

- (a) Fully-Restrained (FR) Moment Connections: A fully-restrained (FR) moment connection transfers moment with a negligible relative 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 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 force-deformation response characteristics of the connection shall be included. The response characteristics of a PR connection shall be documented in the technical literature or established by analytical or experimental means. The component elements of a PR connection shall have sufficient strength, stiffness, and deformation capacity at the strength limit states.

10.2.3.7 Design for serviceability

The overall structure and the individual members, connections and connectors shall be checked for serviceability. Performance requirements for serviceability design are given in Sec 10.12.

10.2.3.8 Design for ponding

The roof system shall be investigated through structural analysis to assure adequate strength and stability under ponding conditions, unless the roof surface is provided with a slope of 20 mm per meter or greater toward points of free drainage or an adequate system of drainage is provided to prevent the accumulation of water. Methods of checking ponding are given in Sec 10.16.

10.2.3.9 Design for fatigue

Fatigue shall be considered in accordance with Sec 10.17, Design for Fatigue, 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 normal building lateral load resisting systems and building enclosure components.

10.2.3.10 Design for fire conditions

Two methods of design for fire conditions are provided in Sec 10.18, Structural Design for Fire Conditions: Qualification Testing and Engineering Analysis. Compliance with the fire protection requirements in Part 4 of this Code shall be required in addition to the requirements of Sec 10.18.

10.2.3.11 Design for corrosion effects

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

10.2.3.12 Design wall thickness for HSS

The design wall thickness, t , shall be used in calculations involving the wall thickness of hollow structural sections (HSS). The design wall thickness, t , shall be taken equal to 0.93 times the nominal wall thickness for electric-resistance-welded (ERW) HSS and equal to the nominal thickness for submerged-arc-welded (SAW) HSS.

10.2.3.13 Gross and net area determination

10.2.3.13.1 Gross area

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

10.2.3.13.2 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 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 Sec 10.10.3.2, of all holes in the chain, and adding, for each gage space in the chain, the quantity $s^2/(4g)$.

Where,

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

g = transverse center-to-center spacing (gage) between fastener gage lines, 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 the product of the thickness and the total width of material that is removed to form the slot.

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

10.2.4 Classification of Sections for Local Buckling

Sections are classified as compact, noncompact, or slender-element sections. For a section to qualify as compact its flanges must be continuously connected to the web or webs and the width-thickness ratios of its compression elements must not exceed the limiting width-thickness ratios λ_p from Table 6.10.1. If the width- thickness ratio of one or more compression elements exceeds λ_p , but does not exceed λ_r from Table 6.10.1, the section is noncompact. If the width-thickness ratio of any element exceeds λ_r , the section is referred to as a slender-element section.

10.2.4.1 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 nominal dimension.

- (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 taken as the full nominal depth of the section.

10.2.4.2 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 or formed sections, h is the clear distance between flanges less the fillet or corner radius at each flange; h_c is twice the distance from the centroid to the inside face of the compression flange less the fillet or corner radius.
- (b) For webs of built-up sections, h is the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used, and h_c is twice the distance from the centroid to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used; h_p is twice the distance from the plastic neutral axis to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used.
- (c) For flange or diaphragm plates in built-up sections, the width b is the distance between adjacent lines of fasteners or lines of welds.
- (d) For flanges of rectangular hollow structural sections (HSS), the width b is the clear distance between webs less the inside corner radius on each side. For webs of rectangular HSS, h is the clear distance between the flanges less the inside corner radius on each side. If the corner radius is not known, b and h shall be taken as the corresponding outside dimension minus three times the thickness. The thickness, t , shall be taken as the design wall thickness, per Sec 10.2.3.12.
- (e) For tapered flanges of rolled sections, the thickness is the nominal value halfway between the free edge and the corresponding face of the web.

10.2.5 Fabrication, Erection and Quality

Shop drawings, fabrication, shop painting, erection, and quality control shall meet the requirements stipulated in Sec 10.13, Fabrication, Erection, and Quality Control.

10.3 Stability Analysis and Design

This Section addresses general requirements for the stability analysis and design of members and frames of steel buildings and structures.

10.3.1 Stability Design Requirements

Stability shall be provided for the structure as a whole and for each of its elements. Any method that considers the influence of second-order effects (including P- Δ and P- δ effects),

flexural, shear and axial deformations, geometric imperfections, and member stiffness reduction due to residual stresses on the stability of the structure and its elements is permitted. The methods prescribed in this Section and Sec 10.14: Direct Analysis Method, satisfy these requirements. All component and connection deformations that contribute to the lateral displacements shall be considered in the stability analysis. In structures designed by elastic analysis, individual member stability and stability of the structure as a whole are provided jointly by:

- Calculation of the required strengths for members, connections and other elements using one of the methods specified in Sec 10.3.2.2, and
- Satisfaction of the member and connection design requirements in this specification based upon those required strengths.

In structures designed by inelastic analysis, the provisions of Sec 10.15 shall be satisfied.

10.3.1.1 Member stability design requirements

Individual member stability is provided by satisfying the provisions of Sections 10.5 to 10.11. Where elements are designed to function as braces to define the unbraced length of columns and beams, the bracing system shall have sufficient stiffness and strength to control member movement at the braced points. Methods of satisfying this requirement are provided in Sec 10.19.

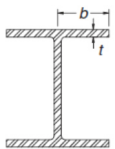
10.3.1.2 System stability design requirements

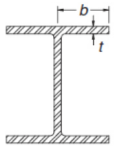
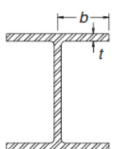
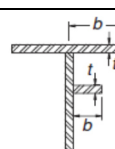
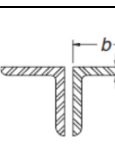
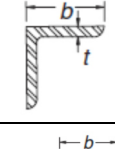
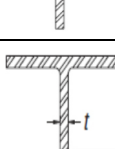
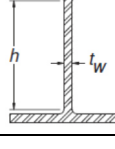
Lateral stability shall be provided by moment frames, braced frames, shear walls, and/or other equivalent lateral load resisting systems. The overturning effects of drift and the destabilizing influence of gravity loads shall be considered. Force transfer and load sharing between elements of the framing systems shall be considered. Braced-frame and shear-wall systems, moment frames, gravity framing systems, and combined systems shall satisfy the following specific requirements:

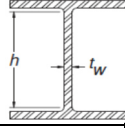
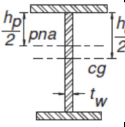
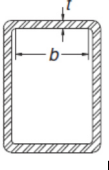
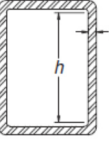
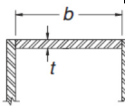
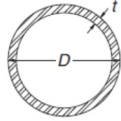
10.3.1.2.1 Braced-frame and shear-wall systems

In structures where lateral stability is provided solely by diagonal bracing, shear walls, or equivalent means, the effective length factor, K , for compression members shall be taken as 1.0, unless structural analysis indicates a smaller value appropriate. In braced-frame systems, it is permitted to design columns, beams, and diagonal members as a vertically cantilevered, simply connected truss.

Table 6.10.1: Limiting Width-Thickness Ratios for Compression Elements

	Case	Description of Element	Width Thickness Ratio	Limiting Width Thickness Ratio		Example
				λ_p (Compact)	λ_r (Noncompact)	
Unstiffened Element	1	Flexure in flanges of rolled I-shaped sections and channels	b/t	$0.38 \sqrt{E/F_y}$	$1.0 \sqrt{E/F_y}$	

	Case	Description of Element	Width Thick- ness Ratio	Limiting Width Thickness Ratio		Example
				λ_p (Compact)	λ_r (Noncompact)	
Unstiffened Elements	2	Flexure in flanges of doubly and singly symmetric I-shaped built-up sections	b/t	$0.38 \sqrt{E/F_y}$	$0.95 \sqrt{k_c E/F_y}^{[a]}$], [b]	
	3	Uniform compression in flanges of rolled I-shaped sections, plates projecting from rolled I-shaped sections; Outstanding legs of pairs of angles in continuous contact and flanges of channels	b/t	NA	$0.56 \sqrt{E/F_y}$	
	4	Uniform compression in flanges of built-up I-shaped sections and plates or angle legs projecting from built-up I-shaped section	b/t	NA	$0.64 \sqrt{k_c E/F_y}^{[a]}$]	
	5	Uniform compression in legs of single angles, legs of double angles with separators, and all other unstiffened elements	b/t	NA	$0.45 \sqrt{E/F_y}$	
	6	Flexure in legs of single angles	b/t	$0.54 \sqrt{E/F_y}$	$0.91 \sqrt{E/F_y}$	
	7	Flexure in flanges of tees	b/t	$0.38 \sqrt{E/F_y}$	$1.0 \sqrt{E/F_y}$	
Stiffened Elements	8	Uniform compression in stems of tees	d/t	NA	$0.75 \sqrt{E/F_y}$	
	9	Flexure in webs of doubly symmetric I-shaped sections	h/t_w	$3.76 \sqrt{E/F_y}$	$5.70 \sqrt{E/F_y}$	

	Case	Description of Element	Width Thick- ness Ratio	Limiting Width Thickness Ratio		Example
				λ_p (Compact)	λ_r (Noncompact)	
stiffened Elements	10	Uniform compression in webs of doubly symmetric I-shaped sections	h/t_w	NA	$1.49 \sqrt{E/F_y}$	
	11	Flexure in webs of singly symmetric I-shaped sections	h_c/t_w	$\frac{h_c}{h_p} \sqrt{\frac{E}{F_y}}$ $\frac{(0.54 \frac{M_p}{M_y})^2}{\leq \lambda_r}$	$5.70 \sqrt{E/F_y}$	
	12	Uniform compression in flanges of rectangular box and hollow structural sections of uniform thickness subject to bending or compression; flange cover plates and diaphragm plates between lines of fasteners or welds	b/t	$1.12 \sqrt{E/F_y}$	$1.40 \sqrt{E/F_y}$	
	13	Flexure in webs of rectangular HSS	h/t	$2.42 \sqrt{E/F_y}$	$5.70 \sqrt{E/F_y}$	
	14	Uniform compression in all other stiffened elements	b/t	NA	$1.49 \sqrt{E/F_y}$	
	15	Circular hollow sections in uniform compression in flexure	D/t D/t	NA $0.07 \sqrt{E/F_y}$	$0.11E/F_y$ $0.31E/F_y$	

^(a) $k_c = \frac{4}{\sqrt{h/t_w}}$ But shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes. (See Cases 2 and 4)

^(b) $F_L = 0.7F_y$ For minor-axis bending, major axis bending of slender-web built-up I-shaped members, and major axis bending of compact and noncompact web built-up I-shaped members with $\frac{S_{xt}}{S_{xc}} \geq 0.7$; $F_L = F_y \frac{S_{xt}}{S_{xc}} \geq 0.5F_y$ for major-axis bending of compact and noncompact web built-up I-shaped members with $\frac{S_{xt}}{S_{xc}} < 0.7$ (See Case 2)

10.3.1.2.2 Moment frame systems

In frames where lateral stability is provided by the flexural stiffness of connected beams and columns, the effective length factor K or elastic critical buckling stress, F_e , for columns and beam-columns shall be determined as specified in Sec 10.3.2.

10.3.1.2.3 Gravity framing systems

Columns in gravity framing systems shall be designed based on their actual length ($K = 1.0$) unless analysis shows that a smaller value may be used. The lateral stability of gravity framing systems shall be provided by moment frames, braced frames, shear walls, and/or other equivalent lateral load resisting systems. $P-\Delta$ effects due to load on the gravity columns shall be transferred to the lateral load resisting systems and shall be considered in the calculation of the required strengths of the lateral load resisting systems.

10.3.1.2.4 Combined systems

The analysis and design of members, connections and other elements in combined systems of moment frames, braced frames, and/or shearwalls and gravity frames shall meet the requirements of the respective systems.

10.3.2 Calculation of Required Strengths

Except as permitted in Sec 10.3.2.2.2, required strengths shall be determined using a second-order analysis as specified in Sec 10.3.2.1. Design by either second-order or first-order analysis shall meet the requirements specified in Sec 10.3.2.2.

10.3.2.1 Methods of second-order analysis

Second-order analysis shall conform to the requirements in this Section.

10.3.2.1.1 General second-order elastic analysis

Any second-order elastic analysis method that considers both $P-\Delta$ and $P-\delta$ effects may be used. The Amplified First-Order Elastic Analysis Method defined in Sec 10.3.2.1.2 is an accepted method for second-order elastic analysis of braced, moment, and combined framing systems.

10.3.2.1.2 Second-order analysis by amplified first-order elastic analysis

The following is an approximate second-order analysis procedure for calculating the required flexural and axial strengths in members of lateral load resisting systems. The required second-order flexural strength, M_r , and axial strength, P_r , shall be determined as follows:

$$M_r = B_1 M_{nt} + B_2 M_{lt} \quad (6.10.3a)$$

$$P_r = P_{nt} + B_2 P_{lt} \quad (6.10.3b)$$

Where,

$$B_1 = \frac{C_m}{1 - \alpha P_r / P_{e1}} \geq 1 \quad (6.10.4)$$

For members subjected to axial compression, B_1 may be calculated based on first-order estimate $P_r = P_{nt} + P_{lt}$.

For members in which $B_1 \leq 1.05$, it is conservative to amplify the sum of the non-sway and sway moments (as obtained, for instance, by a first-order elastic analysis) by the B_2 amplifier, in other words, $M_r = B_2 (M_{nt} + M_{lt})$.

$$B_2 = \frac{1}{1 - \frac{\alpha \sum P_{nt}}{\sum P_c^2}} \geq 1 \quad (6.10.5)$$

And, $\alpha = 1.00$ (LRFD) $\alpha = 1.60$ (ASD)

M_r = required second-order flexural strength using LRFD or ASD load combinations, N-mm

M_{nt} = first-order moment using LRFD or ASD load combinations, assuming there is no lateral translation of the frame, N-mm

M_{lt} = first-order moment using LRFD or ASD load combinations caused by lateral translation of the frame only, N-mm

P_r = required second-order axial strength using LRFD or ASD load combinations, N

P_{nt} = first-order axial force using LRFD or ASD load combinations, assuming there is no lateral translation of the frame, N

$\sum P_{nt}$ = total vertical load supported by the story using LRFD or ASD load combinations, including gravity column loads, N

P_{lt} = first-order axial force using LRFD or ASD load combinations caused by lateral translation of the frame only, N

C_m = a coefficient assuming no lateral translation of the frame whose value shall be taken as follows:

For beam-columns not subject to transverse loading between supports in the plane of bending,

$$C_m = 0.6 - 0.4(M_1/M_2) \quad (6.10.6)$$

Where, M_1 and M_2 , calculated from a first-order analysis, are the smaller and larger moments, respectively, at the ends of that portion of the member unbraced in the plane of bending under consideration. M_1 / M_2 is positive when the member is bent in reverse curvature, negative when bent in single curvature.

For beam-columns subjected to transverse loading between supports, the value of C_m shall be determined either by analysis or conservatively taken as 1.0 for all cases.

P_{e1} = elastic critical buckling resistance of the member in the plane of bending, calculated based on the assumption of zero sidesway, N

$$P_{e1} = \frac{\pi^2 EI}{(K_1 L)^2} \quad (6.10.7)$$

$\sum P_{e2}$ = elastic critical buckling resistance for the story determined by sideway buckling analysis, N

For moment frames, where sidesway buckling effective length factors K_2 are determined for the columns, it is permitted to calculate the elastic story sidesway buckling resistance as

$$\sum P_{e2} = \sum \frac{\pi^2 EI}{(K_2 L)^2} \quad (6.10.8a)$$

For all types of lateral load resisting systems, it is permitted to use

$$\sum P_{e2} = R_M \frac{\sum HL}{\Delta_H} \quad (6.10.8b)$$

Where,

E = modulus of elasticity of steel = 200 000 MPa

R_m = 1.0 for braced-frame systems;

= 0.85 for moment-frame and combined systems, unless a larger value is justified by analysis

I = moment of inertia in the plane of bending, mm⁴

L = story height, mm

K_1 = effective length factor in the plane of bending, calculated based on the assumption of no lateral translation, set equal to 1.0 unless analysis indicates that a smaller value may be used

K_2 = effective length factor in the plane of bending, calculated based on a sideway buckling analysis

Δ_H = first-order interstory drift due to lateral forces, mm. Where Δ_H varies over the plan area of the structure, Δ_H shall be the average drift weighted in proportion to vertical load or, alternatively, the maximum drift.

$\sum H$ = story shear produced by the lateral forces used to compute Δ_H , N

10.3.2.2 Design requirements

These requirements apply to all types of braced, moment, and combined framing systems. Where the ratio of second-order drift to first-order drift is equal to or less than 1.5, the required strengths of members, connections and other elements shall be determined by one of the methods specified in Sections 10.3.2.2.1 or 10.3.2.2.2, or by the Direct Analysis Method of Sec 10.14. Where the ratio of second-order drift to first-order drift is greater than 1.5, the required strengths shall be determined by the Direct Analysis Method of Sec 10.14.

For the methods specified in Sections 10.3.2.2.1 or 10.3.2.2.2:

Analyses shall be conducted according to the design and loading requirements specified in either Section 10.2.3.3 (LRFD) or Section 10.2.3.4 (ASD).

The structure shall be analyzed using the nominal geometry and the nominal elastic stiffness for all elements.

10.3.2.2.1 Design by second-order analysis

Where required strengths are determined by a second-order analysis:

- (a) The provisions of Sec 10.3.2.1 shall be satisfied.
- (b) For design by ASD, analyses 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.
- (c) All gravity-only load combinations shall include a minimum lateral load applied at each level of the structure of $0.002Y_i$, where Y_i is the design gravity load applied at level i . This minimum lateral load shall be considered independently in two orthogonal directions.
- (d) Where the ratio of second-order drift to first-order drift is less than or equal to 1.1, members are permitted to be designed using $K = 1.0$. Otherwise, columns and beam-columns in moment frames shall be designed using a K factor or column buckling stress, F_e , determined from a sidesway buckling analysis of the structure. Stiffness reduction adjustment due to column inelasticity is permitted in the determination of the K factor. For braced frames, K for compression members shall be taken as 1.0, unless structural analysis indicates a smaller value may be used.

10.3.2.2.2 Design by first-order analysis

Required strengths are permitted to be determined by a first-order analysis, with all members designed using $K = 1.0$, provided that

- (a) The required compressive strengths of all members whose flexural stiffnesses are considered to contribute to the lateral stability of the structure satisfy the following limitation:

$$\alpha P_r \leq 0.5 P_y \quad (6.10.9)$$

Where,

$$\alpha = 1.0 (\text{LRFD}) \quad \alpha = 1.6 (\text{ASD})$$

P_r = required axial compressive strength under LRFD or ASD load combinations

P_y = member yield strength ($=AF_y$), N.

- (b) All load combinations include an additional lateral load, N_i , applied in combination with other loads at each level of the structure, where

$$N_i = 2.1 \left(\frac{\Delta}{L} \right) Y_i \geq 0.0042 Y_i \quad (6.10.10)$$

Y_i = gravity load from the LRFD load combination or 1.6 times the ASD load combination applied at level i , N

Δ/L = the maximum ratio of Δ to L for all stories in the structure

Δ = first-order interstory drift due to the design loads, mm. Where Δ varies over the plan area of the structure, Δ shall be average drift weighted in proportion to vertical load or, alternatively, maximum drift.

L = story height, mm

This additional lateral load shall be considered independently in two orthogonal directions.

- (c) The non-sway amplification of beam-column moments is considered by applying the B1 amplifier of Sec10.3.2.1 to the total member moments.

10.4 Design of Members for Tension

This Section applies to steel members subject to axial tension caused by static forces acting through the centroidal axis.

10.4.1 Slenderness Limitations

The maximum slenderness (KL/r) limit for design of structural members (except cables and hanger rods) in tension shall be 300 unless it is justified by a comprehensive dynamic analysis (including 2nd order effects if applicable) that a higher slenderness ratio is satisfactory.

Here,

L = laterally unbraced length of the member, mm

r = governing radius of gyration, mm

K = the effective length factor determined in accordance with Sec 10.3.2.

10.4.2 Tensile Strength

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

(a) For tensile yielding in the gross section:

$$P_n = F_y A_g \quad (6.10.11)$$

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

(b) For tensile rupture in the net section:

$$P_n = F_u A_e \quad (6.10.12)$$

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

Where,

A_e = effective net area, mm²

A_g = gross area of member, mm²

F_y = specified minimum yield stress of the type of steel being used, MPa

F_u = specified minimum tensile strength of the type of steel being used, MPa

When members without holes are fully connected by welds, the effective net area used in Eq.6.10.12 shall be as defined in Sec10.4.3. When holes are present in a member with welded end connections, or at the welded connection in the case of plug or slot welds, the effective net area through the holes shall be used in Eq.6.10.12.

10.4.3 Area Determination

10.4.3.1 Gross area

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

10.4.3.2 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 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 Sec10.10.3.2, of all holes in the chain, and adding, for each gage space in the chain, the quantity $s^2/(4g)$

Where,

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

g = transverse center-to-center spacing (gage) between fastener gage lines, 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 net area across plug or slot welds, the weld metal shall not be considered as adding to net area.

10.4.3.3 Effective net area

The effective area of tension members shall be determined as follows:

$$A_e = A_n U \quad (6.10.13)$$

Where, U is the shear lag factor as obtained from Table 6.10.2.

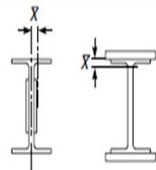
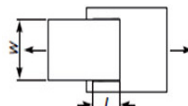
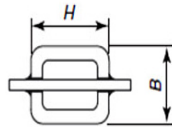
Members such as single angles, double angles and WT sections shall have connections proportioned such that U is equal to or greater than 0.60. Alternatively, a lesser value of U is permitted if these tension members are designed for the effect of eccentricity in accordance with Sections 10.8.1.2 or 10.8.2.

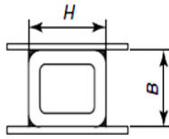
10.4.4 Built-Up Members

For limitations on the longitudinal spacing of connectors between elements in continuous contact consisting of a plate and a shape or two plates, Sec10.10.3.5.

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

Table 6.10.2: Shear Lag Factors for Connections to Tension Members

Case	Description of Element		Shear Lag Factor, U	Example
1	All tension members where the tension load is transmitted directly to each of cross-sectional elements by fasteners or welds. (except as in Cases 3, 4, 5 and 6)		$U = 1.0$	-
2	All tension members, except plates and HSS, where the tension load is transmitted to some but not all of the cross-sectional elements by fasteners or longitudinal welds (Alternatively, for W, M, S and HP, Case 7 may be used.)		$U = 1 - \bar{X}/I$	
3	All tension members where the tension load is transmitted by transverse welds to some but not all of the cross-sectional elements.		$U = 1.0$ and, A_n = area of the directly connected elements	-
4	Plates where the tension load is transmitted by longitudinal welds only.		$I \geq 2w \dots U = 1.0$ $2w > I \geq 1.5w \dots U = 0.87$ $1.5w > I \geq w \dots U = 0.75$	
5	Round HSS with a single concentric gusset plate		$I \geq 1.3D \dots U = 1.0$ $D \leq I < 1.3D \dots U = 1 - \bar{X} = D/\pi$	
6	Rectangular HSS	with a single concentric gusset plate	$I \geq H \dots U = 1 - \bar{X}/L$ $\bar{X} = \frac{B^2 + 2BH}{4(B + H)}$	

Case	Description of Element		Shear Lag Factor, U	Example
		With two side gusset plates	$I \geq H \dots U = 1 - \bar{X}/L$ $\bar{X} = \frac{B^2}{4(B + H)}$	
7	W, M, S or HP Shapes or Tees cut from these shapes. (If U is calculated per Case 2, the larger value is permitted to be used)	With flange connected with 3 or more fasteners per line in direction of loading	$b_f \geq 2/3d \dots U = 0.90$ $b_f \leq 2/3d \dots U = 0.85$	-
		With web connected with 4 or more fasteners per line in the direction of loading	$U = 0.70$	-
8	Single angles (If U is calculated per Case 2, the larger value is permitted to be used)	With 4 or more fasteners per line in direction of loading	$U = 0.80$	-
		With 2 or 3 fasteners per line in the direction of loading	$U = 0.60$	-

I = Length of connection, mm;
 w = plate width, mm;
 $\bar{X}$ = connection eccentricity, mm;
 B = overall width of rectangular HSS member, measured 90° to the plane of the connection, mm;
 H = overall height of rectangular HSS member, measured in the plane of the connection, mm

10.4.5 Pin-Connected Members

10.4.5.1 Tensile strength

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

- (a) For tensile rupture on the net effective area:

$$P_{\eta} = 2tb_{eff}F_u \quad (6.10.14)$$

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

- (b) For shear rupture on the effective area:

$$P_{\eta} = 0.6F_u A_{sf} \quad (6.10.15)$$

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

Where,

$$A_{sf} = 2t(a + d/2), \text{ mm}^2$$

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

$b_{eff} = 2t + 16$, mm but not more than the actual distance from the edge of the hole to the edge of the part measured in the direction normal to the applied force

d = pin diameter, mm

t = thickness of plate, mm

- (c) For bearing on the projected area of the pin, see Sec 10.10.7.

- (d) For yielding on the gross-section, use Eq.6.10.11.

10.4.5.2 Dimensional requirements 1.33 b_{eff}

The pin hole shall be located midway between the edges of the member in the direction normal to the applied force. 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 mm greater than the diameter of the pin.

The width of the plate at the pin hole shall not be less than $(2b_{eff} + d)$ and the minimum extension, a , beyond the bearing end of the pin hole, parallel to the axis of the member, shall not be less than $1.33b_{eff}$

The corners beyond the pin hole are permitted to be cut at 45° to the axis of the member, provided the *net area* beyond the pin hole, on a plane perpendicular to the cut, is not less than that required beyond the pin hole parallel to the axis of the member.

10.4.6 Eyebars

10.4.6.1 Tensile strength

The available tensile strength of eyebars shall be determined in accordance with Sec 10.4.2, with A_g taken as the cross-sectional area of the body.

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

10.4.6.2 Dimensional requirements

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

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

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

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

A thickness of less than 13 mm is permissible only if external nuts are provided to tighten pin plates and filler plates into snug contact. The width from hole edge to plate edge perpendicular to the direction of applied load shall be greater than two-thirds and, for the purpose of calculation, not more than three-fourths times the eyebar body width.

10.5 Design of Members for Compression

This Section addresses members subject to axial compression through the centroidal axis.

10.5.1 General Provisions

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

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

For doubly symmetric and singly symmetric members the limit state of flexural buckling is applicable.

For singly symmetric and unsymmetric members, and certain doubly symmetric members, such as cruciform or built-up columns, the limit states of torsional or flexural-torsional buckling are also applicable.

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

10.5.2 Slenderness Limitations and Effective Length

The effective length factor, K for calculation of column slenderness, KL/r , shall be determined in accordance with Sec 10.3.

Where,

L = laterally unbraced length of the member, mm

r = governing radius of gyration, mm

K = the effective length factor determined in accordance with Sec 10.3.2.

The maximum limit of slenderness, KL/r , for compression members shall be 150 unless a comprehensive analysis including second order effects (including dynamic effects if any) shows that a higher value is justified.

10.5.3 Compressive Strength for Flexural Buckling of Members without Slender Elements

This Section applies to compression members with compact and non-compact sections, as defined in Sec 10.2.4, for uniformly compressed elements.

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

$$P_n = F_{cr} A_g \quad (6.10.16)$$

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

$$(a) \text{ When } \frac{KL}{r} \leq 4.71 \sqrt{\frac{E}{F_y}} \quad (\text{or } F_e \geq 0.44 F_y)$$

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}} \right] F_y \quad (6.10.17)$$

$$(b) \text{ When } \frac{KL}{r} > 4.71 \sqrt{\frac{E}{F_y}} \quad (\text{or } F_e < 0.44 F_y)$$

$$F_{cr} = 0.877 F_e \quad (6.10.18)$$

Where,

F_e = elastic critical buckling stress determined according to Eq.6.10.19, Sec 10.5.4, or the provisions of Sec 10.3.2, as applicable,

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r} \right)^2} \quad (6.10.19)$$

10.5.4 Compressive Strength for Torsional and Flexural-Torsional Buckling of Members without Slender Elements

This section applies to singly symmetric and unsymmetric members, and certain doubly symmetric members, such as cruciform or built-up columns with compact and non compact sections, as defined in Sec 10.2.4 for uniformly compressed elements. These provisions are not required for single angles, which are covered in Sec 10.5.5.

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

$$P_n = F_{cr} A_g \quad (6.10.20)$$

For double-angle and tee-shaped compression members:

$$F_{cr} = \left(\frac{F_{cry} + F_{crz}}{2H} \right) \left[1 - \sqrt{1 - \frac{4F_{cry}F_{crz}H}{(F_{cry} + F_{crz})^2}} \right] \quad (6.10.21)$$

Where,

F_{cry} is taken as F_{cr} from Eq.6.10.17 or 6.10.18, for flexural buckling about the y-axis of symmetry and $\frac{KL}{r} = \frac{KL}{r_y}$, and

$$F_{crz} = \frac{GJ}{A_g \bar{r}_0^2} \quad (6.10.22)$$

For all other cases, F_{cr} shall be determined according to Eq.6.10.17 or 6.10.18, using the torsional or flexural-torsional elastic buckling stress, F_e determined as follows:

For doubly symmetric members:

$$F_e = \left[\frac{\pi^2 E C_w}{(K_z L)^2} + GJ \right] \frac{1}{I_x + I_y} \quad (6.10.23)$$

For singly symmetric members where y is the axis of symmetry:

$$F_e = \left(\frac{F_{ey} + F_{ez}}{2H} \right) \left[1 - \sqrt{1 - \frac{4F_{ey}F_{ez}H}{(F_{ey} + F_{ez})^2}} \right] \quad (6.10.24)$$

For unsymmetric members, F_e is the lowest root of the cubic equation:

$$(F_e - F_{ex})(F_e - F_{ey})(F_e - F_{ez}) - F_e^2(F_e - F_{ey})\left(\frac{x_0}{\bar{r}_0}\right)^2 - F_e^2(F_e - F_{ex})\left(\frac{y_0}{\bar{r}_0}\right)^2 = 0 \quad (6.10.25)$$

Where,

A_g = gross area of member, mm²

C_w = warping constant, mm⁶

$$\bar{r}_0^2 = x_0^2 + y_0^2 + \frac{I_x + I_y}{A_g} \quad (6.10.26)$$

$$H = 1 - \frac{x_0^2 + y_0^2}{\bar{r}_0^2} \quad (6.10.27)$$

$$F_{ex} = \frac{\pi^2 E}{\left(\frac{K_x L}{r_x}\right)^2} \quad (6.10.28)$$

$$F_{ey} = \frac{\pi^2 E}{\left(\frac{K_y L}{r_y}\right)^2} \quad (6.10.29)$$

$$F_{ez} = \left(\frac{\pi^2 E C_w}{(K_z L)^2} + GJ \right) \frac{1}{A_g \bar{r}_0^2} \quad (6.10.30)$$

G = shear modulus of elasticity of steel = 77200 MPa

I_x, I_y = moment of inertia about the principal axes, mm⁴

J = torsional constant, mm⁴

K_z = effective length factor for torsional buckling

x_0, y_0 = coordinates of shear center with respect to the centroid, mm

$\bar{r}_0$ = polar radius of gyration about the shear center, mm

r_y = radius of gyration about y-axis, mm

10.5.5 Single Angle Compression Members

The nominal compressive strength, P_n , of single angle members shall be determined in accordance with Sec 10.5.3 or Sec 10.5.7, as appropriate, for axially loaded members, as well as those subject to the slenderness modification of Sec 10.5.5(a) or 10.5.5(b), provided the members meet the criteria imposed.

The effects of eccentricity on single angle members are permitted to be neglected when the members are evaluated as axially loaded compression members using one of the effective slenderness ratios specified below, provided that: (1) members are loaded at the ends in compression through the same one leg; (2) members are attached by welding or by minimum two-bolt connections; and (3) there are no intermediate transverse loads.

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

(i) When $0 \leq \frac{L}{r_x} \leq 80$

$$\frac{KL}{r} = 72 + \frac{0.75L}{r_x} \quad (6.10.31)$$

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

$$\frac{KL}{r} = 32 + \frac{1.25L}{r_x} \leq 200 \quad (6.10.32)$$

For unequal-leg angles with leg length ratios less than 1.7 and connected through the shorter leg, KL/r from Eq.6.10.31 and Eq.6.10.32 shall be increased by adding $4[(b_l/b_s)^2 - 1]$, but KL/r of the members shall not be less than $0.95L/r_z$.

- (b) For equal-leg angles or unequal-leg angles connected through the longer leg that are web members of box or space trusses with adjacent web members attached to the same side of the gusset plate or chord:

(i) When $0 \leq \frac{L}{r_x} \leq 75$

$$\frac{KL}{r} = 60 + \frac{0.8L}{r_x} \quad (6.10.33a)$$

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

$$\frac{KL}{r} = 45 + \frac{L}{r_x} \leq 200 \quad (6.10.33b)$$

For unequal-leg angles with leg length ratios less than 1.7 and connected through the shorter leg, KL/r from Eq.6.10.33a and 6.10.33b shall be increased by adding $6[(b_l/b_s)^2 - 1]$, but KL/r of the members shall not be less than $0.82L/r_z$.

Where,

L = length of member between work points at truss chord centerlines, mm

b_l = longer leg of angle, mm

b_s = shorter leg of angle, mm

r_x = radius of gyration about geometric axis parallel to connected leg, mm

r_z = radius of gyration for the minor principal axis, mm

- (c) Single angle members with different end conditions from those described in Sections 10.5.5(a) or (b), with leg length ratios greater than 1.7, or with transverse loading shall be evaluated for combined axial load and flexure using the provisions of Sec 10.8. End connection to different legs on each end or to both legs, the use of single bolts or the attachment of adjacent web members to opposite sides of the gusset plate or chord shall constitute different end conditions requiring the use of Sec 10.8 provisions.

10.5.6 Built-up Members

10.5.6.1 Compressive Strength

- (a) The nominal compressive strength of built-up members composed of two or more shapes that are interconnected by bolts or welds shall be determined in accordance with Sections 10.5.3, 10.5.4, or 10.5.7 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, KL/r is replaced by $(KL/r)_m$ determined as follows:

- (i) For intermediate connectors that are snug-tight bolted:

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_0^2 + \left(\frac{a}{r_i}\right)^2} \quad (6.10.34)$$

- (ii) For intermediate connectors that are welded or pretensioned bolted:

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_0^2 + 0.82 \frac{\alpha^2}{(1+\alpha^2)} \left(\frac{a}{r_{ib}}\right)^2} \quad (6.10.35)$$

Where,

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

$\left(\frac{KL}{r}\right)_0$ = column slenderness of built-up member acting as a unit in the buckling direction being considered

a = distance between connectors, mm

r_i = minimum radius of gyration of individual component, mm

r_{ib} = radius of gyration of individual component relative to its centroidal axis parallel to member axis of buckling, mm

α = separation ratio = $h/2r_{ib}$

H = distance between centroids of individual components perpendicular to the member axis of buckling, mm

- (b) The nominal compressive strength of built-up members composed of two or more shapes or plates with at least one open side interconnected by perforated cover plates or lacing with tie plates shall be determined in accordance with Sections 10.5.3, 10.5.4, or 10.5.7 subject to modification given in Sec 10.5.6.1 (a).

10.5.6.2 Dimensional requirements

Individual components of compression members composed of two or more shapes shall be connected to one another at intervals, a , such that the effective slenderness ratio k_a/r_i of each of the component shapes, between the fasteners, does not exceed three-fourths times the governing slenderness ratio of the built-up member. The least radius of gyration, r_i , shall be used in computing the slenderness ratio of each component part. The end connection shall be welded or pre-tensioned bolted with Class A or B faying surfaces.

At the ends of built-up compression members bearing on base plates or milled 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 $1\frac{1}{2}$ times the maximum width of the member.

Along the length of built-up compression members between the end connections required above, longitudinal spacing for intermittent welds or bolts shall be adequate to provide for the transfer of the required forces. For limitations on the longitudinal spacing of fasteners between elements in continuous contact consisting of a plate and a shape or two plates, see Sec 10.10.3.5. Where a component of a built-up compression member consists of an outside plate, the maximum spacing shall not exceed the thickness of the thinner outside plate times $0.75\sqrt{E/F_y}$, nor 305 mm, when intermittent welds are provided along the edges of the components or when fasteners are provided on all gage lines at each section. When fasteners are staggered, the maximum spacing on each gage line shall not exceed the thickness of thinner outside plate times $1.12\sqrt{E/F_y}$ nor 460 mm.

Open sides of compression members built up from plates or shapes shall be provided with continuous cover plates perforated with a succession of access holes. The unsupported width of such plates at access holes, as defined in Sec 10.2.4, is assumed to contribute to the available strength provided the following requirements are met:

- (1) The width-thickness ratio shall conform to the limitations of Sec 10.2.4.
- (2) The ratio of length (in direction of stress) to width of hole shall not exceed two.
- (3) The clear distance between holes in the direction of stress shall be not less than the transverse distance between nearest lines of connecting fasteners or welds.
- (4) The periphery of the holes at all points shall have a minimum radius of 38 mm.

As an alternative to perforated cover plates, lacing with tie plates is permitted at each end and at intermediate points if the lacing is interrupted. Tie plates shall be as near the ends as practicable. In members providing available strength, the end tie plates shall have a

length of not less than the distance between the lines of fasteners or welds connecting them to the components of the member. Intermediate tie plates shall have a length not less than one-half of this distance. The thickness of tie plates shall be not less than one-fiftieth of the distance between lines of welds or fasteners connecting them to the segments of the members. In welded construction, the welding on each line connecting a tie plate shall total not less than one-third the length of the plate. In bolted construction, the spacing in the direction of stress in tie plates shall be not more than six diameters and the tie plates shall be connected to each segment by at least three fasteners.

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

For additional spacing requirements, see Sec 10.10.3.5.

10.5.7 Members with Slender Elements

This Section applies to compression members with slender sections, as defined in Sec 10.2.4 for uniformly compressed elements.

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

$$P_n = F_{cr} A_g \quad (6.10.36)$$

$$\text{When } \frac{KL}{r} \leq 4.71 \sqrt{\frac{E}{QF_y}} \quad (\text{or } F_e \geq 0.44 QF_y)$$

$$F_{cr} = Q \left[0.658^{\frac{QF_y}{F_e}} \right] F_y \quad (6.10.37)$$

$$\text{When } \frac{KL}{r} > 4.71 \sqrt{\frac{E}{QF_y}} \quad (\text{or } F_e < 0.44 QF_y)$$

$$F_{cr} = 0.877 F_e \quad (6.10.38)$$

Where,

F_e = elastic critical buckling stress, calculated using Eq.6.10.19 and 6.10.23 for doubly symmetric members, Eq.6.10.19 and 6.10.24 for singly symmetric members, and Eq.6.10.25 for unsymmetric members, except for single angles where F_e is calculated using Eq.6.10.19.

$Q = 1.0$ for members with compact and noncompact sections, as defined in Sec 10.2.4, for uniformly compressed elements.

$= Q_s Q_a$ for members with slender-element sections, as defined in Sec 10.2.4, for uniformly compressed elements.

10.5.7.1 Slender unstiffened elements, Q_s

The reduction factor Q_s for slender unstiffened elements is defined as follows:

- (a) For flanges, angles, and plates projecting from rolled columns or other compression members:

$$\text{When } \frac{b}{t} \leq 0.56 \sqrt{\frac{E}{F_y}}$$

$$Q_s = 1.0 \quad (6.10.39)$$

$$\text{When } 0.56 \sqrt{E/F_y} < b/t < 1.03 \sqrt{E/F_y}$$

$$Q_s = 1.415 - 0.74 \left(\frac{b}{t} \right) \sqrt{\frac{F_y}{E}} \quad (6.10.40)$$

$$\text{When } b/t \geq 1.03 \sqrt{E/F_y}$$

$$Q_s = \frac{0.69E}{F_y \left(\frac{b}{t} \right)^2} \quad (6.10.41)$$

- (b) For flanges, angles, and plates projecting from built-up columns or other compression members:

$$\text{When } \frac{b}{t} \leq 0.64 \sqrt{\frac{Ek_c}{F_y}}$$

$$Q_s = 1.0 \quad (6.10.42)$$

When $0.64\sqrt{\frac{Ek_c}{F_y}} < b/t \leq 1.17\sqrt{\frac{Ek_c}{F_y}}$

$$Q_s = 1.415 - 0.65\left(\frac{b}{t}\right)\sqrt{\frac{F_y}{Ek_c}} \quad (6.10.43)$$

When $\frac{b}{t} > 1.17\sqrt{\frac{Ek_c}{F_y}}$

$$Q_s = \frac{0.90Ek_c}{F_y\left(\frac{b}{t}\right)^2} \quad (6.10.44)$$

Where, $k_c = \frac{4}{\sqrt{h/t_w}}$, and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes.

(c) For single angles

When $\frac{b}{t} \leq 0.45\sqrt{\frac{E}{F_y}}$

$$Q_s = 1.0 \quad (6.10.45)$$

When $0.45\sqrt{E/F_y} < b/t \leq 0.91\sqrt{E/F_y}$

$$Q_s = 1.34 - 0.76\left(\frac{b}{t}\right)\sqrt{\frac{F_y}{E}} \quad (6.10.46)$$

When $b/t > 0.91\sqrt{E/F_y}$

$$Q_s = \frac{0.53E}{F_y\left(\frac{b}{t}\right)^2} \quad (6.10.47)$$

Where, b = full width of longest angle leg, mm

(d) For stems of tees

When $\frac{d}{t} \leq 0.75\sqrt{\frac{E}{F_y}}$

$$Q_s = 1.0 \quad (6.10.48)$$

When $0.75 \sqrt{\frac{E}{F_y}} < d/t \leq 1.03 \sqrt{\frac{E}{F_y}}$

$$Q_s = 1.908 - 1.22 \left(\frac{d}{t} \right) \sqrt{\frac{F_y}{E}} \quad (6.10.49)$$

When $d/t > 1.03 \sqrt{\frac{E}{F_y}}$

$$Q_s = \frac{0.69E}{F_y \left(\frac{d}{t} \right)^2} \quad (6.10.50)$$

Where,

b = width of unstiffened compression element, as defined in Sec 10.2.4, mm

d = the full nominal depth of tee, mm

t = thickness of element, mm

10.5.7.2 Slender unstiffened elements, Q_s

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

$$Q_a = \frac{A_{eff}}{A} \quad (6.10.51)$$

A = total cross-sectional area of member, mm²

A_{eff} = summation of effective areas of the cross section based on the reduced effective width, b_e , mm².

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

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

$$b_e = 1.92t \sqrt{\frac{E}{f}} \left[1 - \frac{0.34}{(b/t)} \sqrt{\frac{E}{f}} \right] \leq b \quad (6.10.52)$$

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

- (b) For flanges of square and rectangular slender-element sections of uniform thickness

$$\text{with } \frac{b}{t} \geq 1.40 \sqrt{\frac{E}{f}}$$

$$b_e = 1.92t \sqrt{\frac{E}{f}} \left[1 - \frac{0.38}{(b/t)} \sqrt{\frac{E}{f}} \right] \leq b \quad (6.10.53)$$

Where $f = P_n/A_{eff}$

- (c) For axially-loaded circular sections:

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

$$Q = Q_a = \frac{0.038E}{F_y(D/t)} + \frac{2}{3} \quad (6.10.54)$$

Where,

D = outside diameter, mm

t = wall thickness, mm

10.6 Design of Members for Flexure

This Section applies to members subject to simple bending about one principal axis. For simple bending, the member is loaded in a plane parallel to a principal axis that passes through the shear center or is restrained against twisting at load points and supports. The general provisions are provided in Sec 10.6.1. Various Section properties of members are provided in Table 6.10.3.

10.6.1 General Provisions

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

- (a) For all provisions in this Sec 10.6

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

And, the nominal flexural strength, M_n , shall be determined according to Sections 10.6.2 to 10.6.12.

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

The following terms are common to the Equations in this Chapter except where noted:

C_b = lateral-torsional buckling modification factor for non-uniform moment diagrams when both ends of the unsupported segment are braced

$$C_b = \frac{12.5M_{max}}{2.5M_{max}+3M_A+4M_B+3M_C} R_m \leq 3.0 \quad (6.10.55)$$

Where,

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

M_A = absolute value of moment at quarter point of the unbraced segment, N-mm

M_B = absolute value of moment at centerline of the unbraced segment, N-mm

M_C = absolute value of moment at three-quarter point of the unbraced segment, N-mm

R_m = cross-section monosymmetry parameter

= 1.0, doubly symmetric members

= 1.0, singly symmetric members subjected to single curvature bending

= $0.5 + 2 \left(\frac{I_{yc}}{I_y} \right)^2$, singly symmetric members subjected to reverse curvature bending

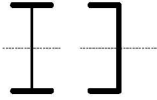
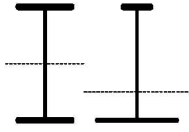
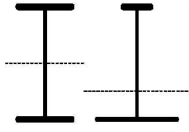
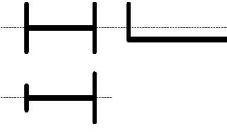
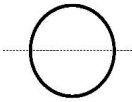
I_y = moment of inertia about the principal y-axis, mm⁴

I_{yc} = moment of inertia about y-axis referred to the compression flange, or if reverse curvature bending, referred to the smaller flange, mm⁴

In singly symmetric members subjected 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.

C_b is permitted to be conservatively taken as 1.0 for all cases. For cantilevers or overhangs where the free end is unbraced, $C_b = 1.0$.

Table 6.10.3: Section Types and Selection Table for the Application of Sub-sections of Sec 10.6

Sub-Section in this provision	Cross Section	Flange Slenderness	Web Slenderness	Limit States
10.6.2		C	C	Y, LTB
10.6.3		NC, S	C	LTB, FLB
10.6.4		C, NC, S	C, NC	Y, LTB, FLB, TFY
10.6.5		C, NC, S	S	Y, LTB, FLB, TFY
10.6.6		C, NC, S	N/A	Y, FLB
10.6.7		C, NC, S	C, NC	Y, FLB, WLB
10.6.8		N/A	N/A	Y, LB

Sub-Section in this provision	Cross Section	Flange Slenderness	Web Slenderness	Limit States
10.6.9		C, NC, S	N/A	Y, LTB, FLB
10.6.10		N/A	N/A	Y, LTB, LLB
10.6.11		N/A	N/A	Y, LTB
10.6.12	Unsymmetrical shapes	N/A	N/A	All limit states
Y = yielding, LTB = lateral-torsional buckling, FLB = flange local buckling, WLB = web local buckling, TFY = tension flange yielding, LLB = leg local buckling, LB = local buckling, C = compact, NC = noncompact, S = slender				

10.6.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent about their Major Axis

This Section applies to doubly symmetric I-shaped members and channels bent about their major axis, having compact webs and compact flanges as defined in Sec10.2.4.

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.

10.6.2.1 Yielding

$$M_n = M_p = F_y Z_x \quad (6.10.56)$$

Where,

F_y = specified minimum yield stress of the type of steel being used, MPa

Z_x = plastic section modulus about the x-axis, mm³

10.6.2.2 Lateral –torsional buckling

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

$$M_n = C_b \left[M_p - (M_p - 0.7F_y S_x) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \leq M_p \quad (6.10.57)$$

(c) When $L_b > L_r$

$$M_n = F_{cr} S_x \leq M_p \quad (6.10.58)$$

Where,

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

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}} \right)^2} \sqrt{1 + 0.078 \frac{Jc}{S_x h_o} \left(\frac{L_b}{r_{ts}} \right)^2} \quad (6.10.59)$$

And where,

 E = modulus of elasticity of steel = 200000 MPa J = torsional constant, mm⁴ S_x = elastic section modulus taken about the x-axis, mm³The limiting lengths L_p and L_r are determined as follows:

$$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} \quad (6.10.60)$$

$$L_r = 1.95 r_{ts} \frac{E}{0.7 F_y} \sqrt{\frac{Jc}{S_x h_o}} \sqrt{1 + \sqrt{1 + 6.76 \left(\frac{0.7 F_y S_x h_o}{E Jc} \right)^2}} \quad (6.10.61)$$

Where,

$$r_{ts}^2 = \frac{\sqrt{I_y C_w}}{S_x} \quad (6.10.62)$$

And,

$$\text{For a doubly symmetric I-shape: } c = 1 \quad (6.10.63a)$$

$$\text{For a channel: } c = \frac{h_o}{2} \sqrt{\frac{I_y}{C_w}} \quad (6.10.63b)$$

Where, h_o = distance between the flange centroids, mm

10.6.3 Doubly Symmetric I-Shaped Members with Compact Webs and Noncompact or Slender Flanges Bent about their Major Axis

This Section applies to doubly symmetric I-shaped members bent about their major axis having compact webs and noncompact or slender flanges as defined in Sec10.2.4.

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.

10.6.3.1 Lateral –torsional buckling

For lateral-torsional buckling, the provisions of Sec 10.6.2.2 shall apply.

10.6.3.2 Compression flange local buckling

(a) For sections with noncompact flanges

$$M_n = \left[M_p - (M_p - 0.7F_y S_x) \left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right] \quad (6.10.64)$$

(b) For sections with slender flanges

$$M_n = \frac{0.9Ek_c S_x}{\lambda^2} \quad (6.10.65)$$

Where,

$$\lambda = \frac{b_f}{2t_f}$$

$\lambda_{pf} = \lambda_p$ is the limiting slenderness for a compact flange, Table 6.10.1

$\lambda_{rf} = \lambda_r$ is the limiting slenderness for a noncompact flange, Table 6.10.1

$k_c = \frac{4}{\sqrt{h/t_w}}$ and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes

10.6.4 Other I-Shaped Members with Compact or Noncompact Webs Bent about their Major Axis

This Section applies to: (a) doubly symmetric I-shaped members bent about their major axis with noncompact webs; and (b) singly symmetric I-shaped members with webs attached to the mid-width of the flanges, bent about their major axis, with compact or noncompact webs, as defined in Section 10.2.4.

The nominal flexural strength, M_n , shall be the lowest value obtained according to the limit states of compression flange yielding, lateral-torsional buckling, compression flange local buckling and tension flange yielding.

10.6.4.1 Compression flange yielding

$$M_n = R_{pc}M_{yc} = R_{pc}F_yS_{xc} \quad (6.10.66)$$

10.6.4.2 Lateral-torsional buckling

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

(b) When $L_p < L_b \leq L_r$

$$M_n = C_b \left[R_{pc}M_{yc} - (R_{pc}M_{yc} - F_yS_{xc}) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \leq R_{pc}M_{yc} \quad (6.10.67)$$

(c) When $L_b > L_r$

$$M_n = F_{cr}S_{xc} \leq R_{pc}M_{yc} \quad (6.10.68)$$

Where

$$M_{yc} = F_yS_{xc} \quad (6.10.69)$$

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_t} \right)^2} \sqrt{1 + 0.078 \frac{J}{S_{xc} h_0} \left(\frac{L_b}{r_t} \right)^2} \quad (6.10.70)$$

For, $\frac{I_{yc}}{I_y} \leq 0.23$, J shall be taken as zero.

The stress, F_L , is determined as follows:

(i) For $\frac{S_{xt}}{S_{xc}} \geq 0.7$

$$F_L = 0.7F_y \quad (6.10.71a)$$

(ii) For $\frac{S_{xt}}{S_{xc}} < 0.7$

$$F_L = F_y \frac{S_{xt}}{S_{xc}} \geq 0.5F_y \quad (6.10.71b)$$

The limiting laterally unbraced length for the limit state of yielding, L_p is,

$$L_p = 1.1r_t \sqrt{\frac{E}{F_y}} \quad (6.10.72)$$

The limiting unbraced length for the limit state of inelastic lateral-torsional buckling, L_r , is

$$L_r = 1.95r_t \frac{E}{F_L} \sqrt{\frac{J}{S_{xc}h_0}} \sqrt{1 + \sqrt{1 + 6.76 \left(\frac{F_L S_{xc}h_0}{E J} \right)^2}} \quad (6.10.73)$$

The web plastification factor, R_{pc} , is determined as follows:

$$(i) \text{ For } \frac{h_c}{t_w} \leq \lambda_{pw}$$

$$R_{pc} = \frac{M_p}{M_{yc}} \quad (6.10.74a)$$

$$(ii) \text{ For } \frac{h_c}{t_w} > \lambda_{pw}$$

$$R_{pc} = \left[\frac{M_p}{M_{yc}} - \left(\frac{M_p}{M_{yc}} - 1 \right) \left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}} \right) \right] \leq \frac{M_p}{M_{yc}} \quad (6.10.74b)$$

Where

$$M_p = Z_x F_y \leq 1.6 S_{xc} F_y$$

S_{xc}, S_{xt} = elastic section modulus referred to tension and compression flanges, respectively, mm³

$$\lambda = \frac{h_c}{t_w}$$

$\lambda_{pw} = \lambda_p$ limiting slenderness for a compact web, Table 6.10.1

$\lambda_{rw} = \lambda_r$ limiting slenderness for a noncompact web, Table 6.10.1

The effective radius of gyration for lateral-torsional buckling, r_t , is determined as follows:

- (i) For I-shapes with a rectangular compression flange:

$$r_t = \frac{b_{fc}}{\sqrt{12 \left(\frac{h_0}{d} + \frac{1}{6} a_w \frac{h^2}{h_0 d} \right)}} \quad (6.10.75)$$

Where,

$$a_w = \frac{h_c t_w}{b_{fc} t_{fc}} \quad (6.10.76)$$

b_{fc} = compression flange width, mm

t_{fc} = compression flange thickness, mm

- (ii) For I-shapes with channel caps or cover plates attached to the compression flange:

r_t = radius of gyration of the flange components in flexural compression plus one-third of the web area in compression due to application of major axis bending moment alone, mm

a_w = the ratio of two times the web area in compression due to application of major axis bending moment alone to the area of the compression flange components.

10.6.4.3 Compression flange local buckling

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

- (b) For sections with noncompact flanges

$$M_n = \left[R_{pc} M_{yc} - (R_{pc} M_{yc} - F_L S_{xc}) \left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right] \quad (6.10.77)$$

- (c) For sections with slender flanges

$$M_n = \frac{0.9 E k_c S_{xc}}{\lambda^2} \quad (6.10.78)$$

Where,

F_L is defined in Eq. 6.10.71a and Eq. 6.10.72b

R_{pc} = is the web plastification factor, determined by Eq. 6.10.74

$k_c = \frac{4}{\sqrt{h/t_w}}$ and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes

$$\lambda = \frac{b_{fc}}{2t_{fc}}$$

$\lambda_{pf} = \lambda_p$ limiting slenderness for a compact flange, Table 6.10.1

$\lambda_{rf} = \lambda_r$ limiting slenderness for a noncompact flange, Table 6.10.1

10.6.4.4 Tension flange yielding

(a) When $S_{xt} \geq S_{xc}$, the limit state of tension flange yielding does not apply.

(b) When $S_{xt} < S_{xc}$

$$M_n = R_{pt} M_{yt} \quad (6.10.79)$$

Where,

$$M_{yt} = F_y S_{xt}$$

The web plastification factor corresponding to the tension flange yielding limit state, R_{pt} , is determined as follows:

(i) For $\frac{h_c}{t_w} \leq \lambda_{pw}$

$$R_{pt} = \frac{M_p}{M_{yt}} \quad (6.10.80a)$$

(ii) For $\frac{h_c}{t_w} > \lambda_{pw}$

$$R_{pt} = \left[\frac{M_p}{M_{yt}} - \left(\frac{M_p}{M_{yt}} - 1 \right) \left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}} \right) \right] \leq \frac{M_p}{M_{yt}} \quad (6.10.80b)$$

Where,

$$\lambda = \frac{h_c}{t_w}$$

$\lambda_{pw} = \lambda_p$, the limiting slenderness for a compact web, defined in Table 6.10.1

$\lambda_{rw} = \lambda_r$, the limiting slenderness for a noncompact web, defined in Table 6.10.1

10.6.5 Doubly Symmetric and Singly Symmetric I-Shaped Members with Slender Webs Bent about Major Axis

This Section applies to doubly symmetric and singly symmetric I-shaped members with slender webs attached to the mid-width of the flanges, bent about their major axis, as defined in Sec10.2.4.

The nominal flexural strength M_n , shall be the lowest value obtained according to the limit states of compression flange yielding, lateral-torsional buckling, compression flange local buckling and tension flange yielding.

10.6.5.1 Compression flange yielding

$$M_n = R_{pg} F_y S_{xc} \quad (6.10.81)$$

10.6.5.2 Lateral-torsional buckling

$$M_n = R_{pg} F_{cr} S_{xc} \quad (6.10.82)$$

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

(b) When $L_p < L_b \leq L_r$

$$F_{cr} = C_b \left[F_y - (0.3 F_y) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \leq F_y \quad (6.10.83)$$

(c) When $L_b > L_r$

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_t} \right)^2} \leq F_y \quad (6.10.84)$$

Where,

L_p is defined by Eq.6.10.72

$$L_r = \pi r_t \sqrt{\frac{E}{0.7 F_y}} \quad (6.10.85)$$

R_{pg} is the bending strength reduction factor:

$$R_{pg} = 1 - \frac{a_w}{1200 + 300 a_w} \left(\frac{h_c}{t_w} - 5.7 \sqrt{\frac{E}{F_y}} \right) \leq 1.0 \quad (6.10.86)$$

a_w is defined by Eq.6.10.76 but shall not exceed 10 and

r_t is the effective radius of gyration for lateral buckling as defined in Sec10.6.4.

10.6.5.3 Compression flange local buckling

$$M_n = R_{pg} F_{cr} S_{xc} \quad (6.10.87)$$

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

$$F_{cr} = \left[F_y - (0.3 F_y) \left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right] \quad (6.10.88)$$

- (c) For sections with slender flanges

$$F_{cr} = \frac{0.9 E k_c}{\left(\frac{b_f}{2 t_f} \right)^2} \quad (6.10.89)$$

Where,

$k_c = \frac{4}{\sqrt{h/t_w}}$, and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes

$$\lambda = \frac{b_{fc}}{2 t_{fc}}$$

$\lambda_{pf} = \lambda_p$, the limiting slenderness for a compact flange, Table 6.10.1

$\lambda_{rf} = \lambda_r$, the limiting slenderness for a noncompact flange, Table 6.10.1

10.6.5.4 Tension flange yielding

- (a) When $S_{xt} \geq S_{xc}$, the limit state of tension flange yielding does not apply.
- (b) When $S_{xt} < S_{xc}$

$$M_n = F_y S_{xt} \quad (6.10.90)$$

10.6.6 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.

10.6.6.1 Yielding

$$M_n = M_p = F_y Z_y \leq 1.6 F_y S_y \quad (6.10.91)$$

10.6.6.2 Flange local buckling

- (a) For sections with compact flanges the limit state of yielding shall apply.
 (b) For sections with noncompact flanges

$$M_n = \left[M_p - (M_p - 0.7 F_y S_y) \left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right] \quad (6.10.92)$$

- (c) For sections with slender flanges

$$M_n = F_{cr} S_y \quad (6.10.93)$$

Where,

$$F_{cr} = \frac{0.69E}{\left(\frac{b_f}{2t_f} \right)^2} \quad (6.10.94)$$

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

$\lambda_{pf} = \lambda_p$, the limiting slenderness for a compact flange, Table 6.10.1

$\lambda_{rf} = \lambda_r$, the limiting slenderness for a noncompact flange, Table 6.10.1

S_y for a channel shall be taken as the minimum section modulus

10.6.7 Square and Rectangular HSS and Box-Shaped Members

This section applies to square and rectangular HSS, and doubly symmetric box-shaped members bent about either axis, having compact or noncompact webs and compact, noncompact or slender flanges as defined in Sec10.2.4.

The nominal flexural strength, M_n , shall be the lowest value obtained according to the limit states of yielding (plastic moment), flange local buckling and web local buckling under pure flexure.

10.6.7.1 Yielding

$$M_n = M_p = F_y Z \quad (6.10.95)$$

Where,

Z = plastic section modulus about the axis of bending, mm^3

10.6.7.2 Flange local buckling

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

(b) For sections with noncompact flanges

$$M_n = M_p - (M_p - F_y S) \left(3.57 \frac{b}{t} \sqrt{\frac{F_y}{E}} - 4.0 \right) \leq M_p \quad (6.10.96)$$

(b) For sections with slender flanges

$$M_n = F_y S_{eff} \quad (6.10.97)$$

Where, S_{eff} is the effective section modulus determined with the effective width of the compression flange taken as:

$$b_e = 1.92t \sqrt{\frac{E}{F_y}} \left[1 - \frac{0.38}{b/t} \sqrt{\frac{E}{F_y}} \right] \leq b \quad (6.10.98)$$

10.6.7.3 Web local buckling

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

(b) For sections with noncompact webs

$$M_n = M_p - (M_p - F_y S_x) \left(0.305 \frac{h}{t_w} \sqrt{\frac{F_y}{E}} - 0.738 \right) \leq M_p \quad (6.10.99)$$

10.6.8 Round HSS

This Section applies to round *HSS* having D/t ratios of less than $\frac{0.45E}{F_y}$

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

10.6.8.1 Yielding

$$M_n = M_p = F_y Z \quad (6.10.100)$$

10.6.8.2 Local buckling

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

$$M_n = \left(\frac{0.021E}{\frac{D}{t}} + F_y \right) S \quad (6.10.101)$$

- (b) For sections with slender walls

$$M_n = F_{cr} S \quad (6.10.102)$$

Where,

$$F_{cr} = \frac{0.33E}{\frac{D}{t}} \quad (6.10.103)$$

S = elastic section modulus, mm³

10.6.9 Tees and Double Angles Loaded in the Plane of Symmetry

This Section applies to tees and double angles loaded in the plane of symmetry. The nominal flexural strength, M_n , shall be the lowest value obtained according to the limit states of yielding (plastic moment), lateral-torsional buckling and flange local buckling.

10.6.9.1 Yielding

$$M_n = M_p \quad (6.10.104)$$

Where,

$$M_p = F_y Z_x \leq 1.6 M_y \text{ for stems in tension} \quad (6.10.105)$$

$$\leq M_y \text{ for stems in compression} \quad (6.10.106)$$

10.6.9.2 Lateral-torsional buckling

$$M_n = M_{cr} = \frac{\pi \sqrt{EI_y GJ}}{L_b} [B + \sqrt{1 + B^2}] \quad (6.10.107)$$

Where,

$$B = \pm 2.3 \left(\frac{d}{L_b} \right) \sqrt{\frac{I_y}{J}} \quad (6.10.108)$$

The plus sign for B applies when the stem is in tension and the minus sign applies when the stem is in compression. If the tip of the stem is in compression anywhere along the unbraced length, the negative value of B shall be used.

10.6.9.3 Flange local buckling of tees

$$M_n = F_{cr} S_{xc} \quad (6.10.109)$$

S_{xc} is the elastic section modulus referred to the compression flange.

F_{cr} is determined as follows:

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

$$F_{cr} = F_y \left(1.19 - 0.50 \left(\frac{b_f}{2t_f} \right) \sqrt{\frac{F_y}{E}} \right) \quad (6.10.110)$$

- (c) For slender sections

$$F_{cr} = \frac{0.69E}{\left(\frac{b_f}{2t_f} \right)^2} \quad (6.10.111)$$

10.6.10 Single Angle

This Section applies to single angles with and without continuous lateral restraint along their length.

Single angles with continuous lateral-torsional restraint along the length shall be permitted to be designed on the basis of geometric axis (x, y) bending. Single angles without continuous lateral-torsional restraint along the length shall be designed using the provisions for principal axis bending except where the provision for bending about a geometric axis is permitted.

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

10.6.10.1 Yielding

$$M_n = 1.5M_y \quad (6.10.112)$$

Where, M_y = yield moment about the axis of bending, N-mm

10.6.10.2 Lateral-torsional buckling

For single angles without continuous lateral-torsional restraint along the length

(a) When $M_e \leq M_y$

$$M_n = \left(0.92 - \frac{0.17M_e}{M_y}\right) M_e \quad (6.10.113)$$

(b) When $M_e > M_y$

$$M_n = \left(1.92 - 1.17\sqrt{\frac{M_y}{M_e}}\right) M_y \leq 1.5M_y \quad (6.10.114)$$

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

(i) For bending about one of the geometric axes of an equal-leg angle with no lateral-torsional restraint

(a) With maximum compression at the toe

$$M_e = \frac{0.66Eb^4tC_b}{L^2} \left(\sqrt{1 + 0.78\left(\frac{Lt}{b^2}\right)^2} - 1 \right) \quad (6.10.115a)$$

(b) With maximum tension at the toe

$$M_e = \frac{0.66Eb^4tC_b}{L^2} \left(\sqrt{1 + 0.78\left(\frac{Lt}{b^2}\right)^2} + 1 \right) \quad (6.10.115b)$$

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

(ii) For bending about one of the geometric axes of an equal-leg angle with lateral-torsional restraint at the point of maximum moment only

M_e shall be taken as 1.25 times M_e computed using Eq.6.10.115a or Eq.6.10.115b.

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

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

$$M_e = \frac{0.46Eb^2t^2C_b}{L} \quad (6.10.116)$$

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

$$M_e = \frac{4.9EI_z C_b}{L^2} \left(\sqrt{\beta_w^2 + 0.052 \left(\frac{Lt}{r_z} \right)^2} + \beta_w \right) \quad (6.10.117)$$

Where,

C_b is computed using Eq.6.10.55 with a maximum value of 1.5.

L = laterally unbraced length of a member, mm

I_z = minor principal axis moment of inertia, mm⁴

r_z = radius of gyration for the minor principal axis, mm

t = angle leg thickness, mm

β_w = a section property for unequal leg angles, positive for short legs in compression and negative for long legs in compression. If the long leg is in compression anywhere along the unbraced length of the member, the negative value of β_w shall be used.

10.6.10.3 Leg local buckling

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

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

(b) For sections with noncompact legs:

$$M_n = F_y S_c \left(2.43 - 1.72 \left(\frac{b}{t} \right) \sqrt{\frac{F_y}{E}} \right) \quad (6.10.118)$$

(c) For sections with slender legs

$$M_n = F_{cr} S_c \quad (6.10.119)$$

Where,

$$F_{cr} = \frac{0.71E}{\left(\frac{b}{t} \right)^2} \quad (6.10.120)$$

b = outside width of leg in compression, mm

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

10.6.11 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, as required.

10.6.11.1 Yielding

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

$$M_n = M_p = F_y Z \leq 1.6 M_y \quad (6.10.121)$$

10.6.11.2 Lateral-torsional buckling

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

$$M_n = C_b \left[1.52 - 0.274 \left(\frac{L_b d}{t^2} \right) \frac{F_y}{E} \right] M_y \leq M_p \quad (6.10.122)$$

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

$$M_n = F_{cr} S_x \leq M_p \quad (6.10.123)$$

Where,

$$F_{cr} = \frac{1.9EC_b}{\frac{L_b d}{t^2}} \quad (6.10.124)$$

t = width of rectangular bar parallel to axis of bending, mm

d = depth of rectangular bar, mm

L_b = length between points that are either braced against lateral displacement of the compression region or braced against twist of the cross section, mm

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

10.6.12 Unsymmetrical Shapes

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

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

$$M_n = F_n S \quad (6.10.125)$$

Where, S = lowest elastic section modulus relative to the axis of bending, mm^3

10.6.12.1 Yielding

$$F_n = F_y \quad (6.10.126)$$

10.6.12.2 Lateral-torsional buckling

$$F_n = F_{cr} \leq F_y \quad (6.10.127)$$

Where, F_{cr} = buckling *stress* for the section as determined by analysis, MPa

10.6.12.3 Local buckling

$$F_n = F_{cr} \leq F_y \quad (6.10.128)$$

Where, F_{cr} = buckling *stress* for the section as determined by analysis, MPa

10.6.13 Proportions of Beams and Girders

10.6.13.1 Hole reductions

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

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

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

For, $F_u A_{fn} < Y_t F_y A_{fg}$, the nominal flexural strength, M_n , at the location of the holes in the tension flange shall not be taken greater than:

$$M_n = \frac{F_u A_{fn}}{A_{fg}} S_x \quad (6.10.129)$$

Where,

A_{fg} = gross tension flange area, calculated in accordance with the provisions of Sec 10.4.3.1, mm²

A_{fn} = net tension flange area, calculated in accordance with the provisions of Sec 10.4.3.2, mm²

$Y_t = 1.0$ for $F_y/F_u \leq 0.8$
 $= 1.1$ otherwise

10.6.13.2 Proportioning limits for i-shaped members

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

$$0.1 \leq \frac{I_{yc}}{I_y} \leq 0.9 \quad (6.10.130)$$

I-shaped members with slender webs shall also satisfy the following limits:

(a) For $\frac{a}{h} \leq 1.5$

$$\left(\frac{h}{t_w}\right)_{max} = 11.7 \sqrt{\frac{E}{F_y}} \quad (6.10.131)$$

(b) For $\frac{a}{h} > 1.5$

$$\left(\frac{h}{t_w}\right)_{max} = \frac{0.42E}{F_y} \quad (6.10.132)$$

Where, a = clear distance between transverse stiffeners, mm

In unstiffened girders $\frac{h}{t_w}$ shall not exceed 260. The ratio of the web area to the compression flange area shall not exceed 10.

10.6.13.3 Cover plates

Flanges of welded beams or girders may be varied in thickness or width by splicing a series of plates or by the use of cover plates.

The total cross-sectional area of cover plates of bolted girders shall not exceed 70 percent of the total flange area.

High-strength bolts or welds connecting flange to web, or cover plate to flange, shall be proportioned to resist the total horizontal shear resulting from the bending forces on the girder. The longitudinal distribution of these bolts or intermittent welds shall be in proportion to the intensity of the shear.

However, the longitudinal spacing shall not exceed the maximum permitted for compression or tension members in Sec 10.5.6 or 10.4.4, respectively. Bolts or welds connecting flange to web shall also be proportioned to transmit to web any loads applied directly to the flange, unless provision is made to transmit such loads by direct bearing.

Partial-length cover plates shall be extended beyond the theoretical cutoff point and the extended portion shall be attached to the beam or girder by high-strength bolts in a slip-critical connection or fillet welds. The attachment shall be adequate, at the applicable strength given in Sections 10.10.2.2, 10.10.3.8, or 10.2.3.9 to develop the cover plate's portion of the flexural strength in the beam or girder at the theoretical cutoff point.

For welded cover plates, the welds connecting the cover plate termination to the beam or girder shall have continuous welds along both edges of the cover plate in the length a' , defined below, and shall be adequate to develop the cover plate's portion of the strength of the beam or girder at the distance a' from the end of the cover plate.

- (a) When there is a continuous weld equal to or larger than three-fourths of the plate thickness across the end of the plate

$$a' = w \quad (6.10.133)$$

Where, w = width of cover plate, mm.

- (b) When there is a continuous weld smaller than three-fourths of the plate thickness across the end of the plate

$$a' = 1.5w \quad (6.10.134)$$

- (c) When there is no weld across the end of the plate

$$a' = 2w \quad (6.10.135)$$

10.6.13.4 Built-up beams

Where two or more beams or channels are used side-by-side to form a flexural member, they shall be connected together in compliance with Sec 10.5.6.2. When concentrated loads are carried from one beam to another, or distributed between the beams, diaphragms having sufficient stiffness to distribute the load shall be welded or bolted between the beams.

10.7 Design Of Members For Shear

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

10.7.1 General Provisions

Two methods of calculating shear strength are presented below. The method presented in Sec 10.7.2 does not utilize the post buckling strength of the member (tension field action). The method presented in Sec 10.7.3 utilizes tension field action.

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

For all provisions in this Section except Sec 10.7.2.1(a):

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

10.7.2 Members with Unstiffened or Stiffened Webs

10.7.2.1 Nominal shear strength

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

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

$$V_n = 0.6F_y A_w C_w \quad (6.10.136)$$

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

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

And,

$$C_v = 1.0 \quad (6.10.137)$$

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

(i) For $h/t_w \leq 1.10\sqrt{k_v E/F_y}$

$$C_v = 1.0 \quad (6.10.138)$$

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

$$C_v = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} \quad (6.10.139)$$

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

$$C_v = \frac{1.51Ek_v}{(h/t_w)^2 F_y} \quad (6.10.140)$$

Where

A_w = the overall depth times the web thickness, dt_w , mm²

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

(i) For unstiffened webs with $h/t_w < 260$, $k_v = 5$ except for the stem of tee shapes where, $k_v = 1.2$.

(ii) For stiffened webs,

$$k_v = 5 + \frac{5}{(a/h)^2}$$

$$= 5 \text{ when } a/h > 3.0 \text{ or } a/h > \left[\frac{260}{(h/t_w)} \right]^2$$

Where,

a = clear distance between transverse stiffeners, mm

h = for rolled shapes, the clear distance between flanges less the fillet or corner radii, mm

= for built-up welded sections, the clear distance between flanges, mm

= for built-up bolted sections, the distance between fastener lines, mm

= for tees, the overall depth, mm.

10.7.2.2 Transverse stiffeners

Transverse stiffeners are not required where $h/t_w \leq 2.46\sqrt{E/F_y}$, or where the required shear strength is less than or equal to the available shear strength provided in accordance with Sec 10.7.2.1 for $k_v = 5$.

Transverse stiffeners used to develop the available web shear strength, as provided in Sec 10.7.2.1, shall have a moment of inertia about an axis in the web center for stiffener pairs or about the face in contact with the web plate for single stiffeners, which shall not be less than $at_w^3 j$, where

$$j = \frac{2.5}{(a/h)^2} - 2 \geq 0.5 \quad (6.10.141)$$

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 to the web-to-flange weld. When single stiffeners are used, they shall be attached to the compression flange, if it consists of a rectangular plate, to resist any uplift tendency due to torsion in the flange. When lateral bracing is attached to a stiffener, or a pair of stiffeners, these, in turn, shall be connected to the compression flange to transmit 1 percent of the total flange force, unless the flange is composed only of angles.

Bolts connecting stiffeners to the girder web shall be spaced not more than 305 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 250 mm.

10.7.3 Tension Field Action

10.7.3.1 Limits on the use of tension field action

Consideration of tension field action is permitted for flanged members when the web plate is supported on all four sides by flanges or stiffeners. Consideration of tension field action is not permitted for:

End panels in all members with transverse stiffeners;

Members when a/h exceeds 3.0 or $[260/(h/t_w)]^2$;

$$2 A_w / (A_{fc} + A_{ft}) > 2.5; \text{ or}$$

$$h/b_{fc} \text{ or } h/b_{ft} > 6.0$$

Where,

A_{fc} = area of compression flange, mm²

A_{ft} = area of tension flange, mm²

b_{fc} = width of compression flange, mm

b_{ft} = width of tension flange, mm

In these cases, the nominal shear strength, V_n , shall be determined according to the provisions of Sec 10.7.2.

10.7.3.2 Nominal shear strength with tension field action

When tension field action is permitted according to Sec 10.7.3.1, the nominal shear strength, V_n , with tension field action, according to the limit state of tension field yielding, shall be

For, $h/t_w \leq 1.10\sqrt{k_v E/F_y}$

$$V_n = 0.6F_y A_w \quad (6.10.142)$$

For, $h/t_w > 1.10\sqrt{k_v E/F_y}$

$$V_n = 0.6F_y A_w \left(C_v + \frac{1-C_v}{1.15\sqrt{1+(a/h)^2}} \right) \quad (6.10.143)$$

Where, k_v and C_v are as defined in Sec 10.7.2.1.

10.7.3.3 Transverse stiffeners

Transverse stiffeners subject to tension field action shall meet the requirements of Sec 10.7.2.2 and the following limitations:

$$(b/t)_{st} \leq 0.56 \sqrt{\frac{E}{F_{yst}}}$$

$$A_{st} > \frac{F_y}{F_{yst}} \left[0.15D_s h t_w (1 - C_v) \frac{V_r}{V_c} - 18t_w^2 \right] \geq 0 \quad (6.10.144)$$

Where,

$(b/t)_{st}$ = the width-thickness ratio of the stiffener

F_{yst} = specified minimum yield stress of the stiffener material, MPa

C_v = coefficient defined in Sec 10.7.2.1

D_s = 1.0 for stiffeners in pairs
 = 1.8 for single angle stiffeners
 = 2.4 for single plate stiffeners

V_r = required shear strength at the location of the stiffener, N

V_c = available shear strength; $\phi_v V_n$ (LRFD) or V_n/Ω_v (ASD) with V_n as defined in Sec 10.7.3.2, N

10.7.4 Single Angles

The nominal shear strength, V_n , of a single angle leg shall be determined using Eq. 10.136 with $C_v = 1.0$, $A_w = b_t$ where b = width of the leg resisting the shear force, mm and $k_v = 1.2$.

10.7.5 Rectangular HSS and Box Members

The nominal shear strength, V_n , of rectangular HSS and box members shall be determined using the provisions of Sec 10.7.2.1 with $A_w = 2ht$ where h for the width resisting the shear force shall be taken as the clear distance between the flanges less the inside corner radius on each side and $t_w = t$ and $k_y = 5$. If the corner radius is not known, h shall be taken as the corresponding outside dimension minus three times the thickness.

10.7.6 Round HSS

The nominal shear strength, V_n of round HSS, according to the limit states of shear yielding and shear buckling, is

$$V_n = F_{cr} A_g / 2 \quad (6.10.145)$$

Where,

F_{cr} shall be the larger of

$$F_{cr} = \frac{1.60E}{\sqrt{\frac{L_v}{D} \left(\frac{D}{t}\right)^{\frac{5}{4}}}} \quad (6.10.146a)$$

And,

$$F_{cr} = \frac{0.78E}{\left(\frac{D}{t}\right)^{\frac{3}{2}}} \quad (6.10.146b)$$

But shall not exceed $0.6 F_y$

A_g = gross area of section based on design wall thickness, mm^2

D = outside diameter, mm

L_v = the distance from maximum to zero shear force, mm

T = design wall thickness, equal to 0.93 times the nominal wall thickness for ERW HSS and equal to the nominal thickness for SAW HSS, mm

10.7.7 Weak Axis Shear in Singly and Doubly Symmetric Shapes

For singly and doubly symmetric shapes loaded in the weak axis without torsion, the nominal shear strength, V_n , for each shear resisting element shall be determined using Eq. 6.10.136 and Sec 10.7.2.1b with $A_w = b_f t_f$ and $k_v = 1.2$.

10.7.8 Beams and Girders with Web Openings

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

10.8 Design of Members for Combined Forces and Torsion

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

10.8.1 Doubly and Singly Symmetric Members Subject to Flexure and Axial Force**10.8.1.1 Doubly and singly symmetric members in flexure and compression**

The interaction of flexure and compression in doubly symmetric members and singly symmetric members for which $0.1 \leq \left(\frac{I_{yc}}{I_y}\right) \leq 0.9$, that are constrained to bend about a geometric axis (x and/or y) shall be limited by Equations 6.10.147a and 6.10.147b, where I_{yc} is the moment of inertia about the y-axis referred to the compression flange, mm⁴.

For $\frac{P_r}{P_c} \geq 0.2$

$$\frac{P_r}{P_c} + \frac{8}{9} \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \right) \leq 1.0 \quad (6.10.147a)$$

For $\frac{P_r}{P_c} < 0.2$

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \right) \leq 1.0 \quad (6.10.147b)$$

Where,

P_r = required axial compressive strength, N

P_c = available axial compressive strength, N

M_r = required flexural strength, N-mm

M_c = available flexural strength, N-mm

x = subscript relating symbol to strong axis bending

y = subscript relating symbol to weak axis bending

For design according to Sec 10.2.3.3 (LRFD)

P_r = required axial compressive strength using LRFD load combinations, N

$P_c = \phi_c P_n$ = design axial compressive strength, determined in accordance with Sec 10.5, N

M_r = required flexural strength using LRFD load combinations, N-mm

$M_c = \phi_b M_n$ = design flexural strength determined in accordance with Sec 10.6, N-mm

ϕ_c = resistance factor for compression = 0.90

ϕ_b = resistance factor for flexure = 0.90

For design according to Sec 10.2.3.4 (ASD)

P_r = required axial compressive strength using ASD load combinations, N

$P_c = P_n / \Omega_c$ = allowable axial compressive strength, determined in accordance with Sec 10.5, N

M_r = required flexural strength using ASD load combinations, N-mm

$M_c = M_n / \Omega_b$ = allowable flexural strength determined in accordance with Sec 10.6, N-mm

Ω_c = safety factor for compression = 1.67

Ω_b = safety factor for flexure = 1.67

10.8.1.2 Doubly and singly symmetric members in flexure and tension

The interaction of flexure and tension in doubly symmetric members and singly symmetric members constrained to bend about a geometric axis (x and/or y) shall be limited by Eq. 6.10.147a and 6.10.147b,

Where,

For design according to Sec 10.2.3.3 (LRFD)

P_r = required tensile strength using LRFD load combinations, N

$P_c = \phi_t P_n$ = design tensile strength, determined in accordance with Sec 10.4.2, N

M_r = required flexural strength using LRFD load combinations, N-mm

$M_c = \phi_b M_n$ = design flexural strength determined in accordance with Sec 10.6, N-mm

ϕ_t = resistance factor for tension (see Sec 10.4.2)

ϕ_b = resistance factor for flexure = 0.90

For doubly symmetric members, C_b in Sec 10.6 may be increased by $\sqrt{1 + \frac{P_u}{P_{ey}}}$ for axial tension that acts concurrently with flexure,

Where,

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

For design according to Sec 10.2.3.4 (ASD)

P_r = required tensile strength using ASD load combinations, N

$P_c = P_n / \Omega_t$ = allowable tensile strength, determined in accordance with Sec 10.4.2, N

M_r = required flexural strength using ASD load combinations, N-mm

$M_c = M_n / \Omega_b$ = allowable flexural strength determined in accordance with Sec 10.6, N-mm

Ω_t = safety factor for tension (see Sec 10.4.2)

Ω_b = safety factor for flexure = 1.67

For doubly symmetric members, C_b in Sec 10.6 may be increased by $\sqrt{1 + \frac{1.5P_a}{P_{ey}}}$ for axial tension that acts concurrently with flexure.

Where,

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

A more detailed analysis of the interaction of flexure and tension is permitted in lieu of Equations 6.10.147a and 6.10.147b.

10.8.1.3 Doubly symmetric members in single axis flexure and compression

For doubly symmetric members in flexure and compression with moments primarily in one plane, it is permissible to consider the two independent limit states, in-plane instability and

out-of-plane buckling or flexural-torsional buckling, separately in lieu of the combined approach provided in Sec 10.8.1.1.

- (a) For the limit state of in-plane instability, Eq.6.10.147 shall be used with P_c , M_r , and M_c determined in the plane of bending.
- (b) For the limit state of out-of-plane buckling

$$\frac{P_r}{P_{co}} + \left(\frac{M_r}{M_{cx}} \right)^2 \leq 1.0 \quad (6.10.148)$$

Where,

P_{co} = available compressive strength out of the plane of bending, N

M_{cx} = available flexural-torsional strength for strong axis flexure determined from Sec 10.6, N-mm

If bending occurs only about the weak axis, the moment ratio in Eq.6.10.148 shall be neglected.

For members with significant biaxial moments, $\left(\frac{M_r}{M_c} \right) \geq 0.05$ in both directions, the provisions of Sec 10.8.1.1 shall be followed.

10.8.2 Unsymmetric and Other Members Subject to Flexure and Axial Force

This Section addresses the interaction of flexure and axial *stress* for shapes not covered in Sec 10.8.1. It is permitted to use the provisions of this Section for any shape in lieu of the provisions of Sec 10.8.1.

$$\left| \frac{f_a}{F_a} + \frac{f_{bw}}{F_{bw}} + \frac{f_{bz}}{F_{bz}} \right| \leq 1.0 \quad (6.10.149)$$

Where,

f_a = required axial stress at the point of consideration, MPa

F_a = available axial stress at the point of consideration, MPa

f_{bw}, f_{bz} = required flexural stress at the point of consideration, MPa

F_{bw}, F_{bz} = available flexural stress at the point of consideration, MPa

w = subscript relating symbol to major principal axis bending

z = subscript relating symbol to minor principal axis bending

For design according to Sec 10.2.3.3 (LRFD)

f_a = required axial stress using LRFD load combinations, MPa

$F_a = \phi_c F_{cr}$ = design axial stress, determined in accordance with Sec 10.5 for compression or Sec 10.4.2 for tension, MPa

f_{bw}, f_{bz} = required flexural stress at the specific location in the cross section using LRFD load combinations, MPa

$F_{bw}, F_{bz} = \frac{\phi_b M_n}{S}$ = design flexural stress determined in accordance with Sec 10.6, MPa. Use the section modulus for the specific location in the cross section and consider the sign of the stress.

ϕ_c = resistance factor for compression = 0.90

ϕ_t = resistance factor for tension (Sec 10.4.2)

ϕ_b = resistance factor for flexure = 0.90

For design according to Sec 10.2.3.4 (ASD)

f_a = required axial stress using ASD load combinations, MPa

$F_a = \frac{F_{cr}}{\Omega_c}$ = allowable axial stress determined in accordance with Sec 10.5 for compression, or Sec 10.4.2 for tension, MPa

f_{bw}, f_{bz} = required flexural stress at the specific location in cross section using ASD load combinations, MPa

$F_{bw}, F_{bz} = \frac{M_n}{\Omega_b S}$ = allowable flexural stress determined in accordance with Sec 10.6, MPa. Use the section modulus for the specific location in the cross section and consider the sign of the stress.

Ω_c = safety factor for compression = 1.67

Ω_t = safety factor for tension (Sec 10.4.2)

Ω_b = safety factor for flexure = 1.67

Eq.6.10.149 shall be evaluated using the principal bending axes by considering the sense of the flexural stresses at the critical points of the cross section. The flexural terms are either added to or subtracted from the axial term as appropriate. When the axial *force* is compression, second order effects shall be included according to the provisions of Sec 10.3.

A more detailed analysis of the interaction of flexure and tension is permitted in lieu of Eq.6.10.149.

10.8.3 Members under Torsion and Combined Torsion, Flexure, Shear and/or Axial Force

10.8.3.1 Torsional strength of round and rectangular HSS

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

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

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

$$T_n = F_{cr} C \quad (6.10.150)$$

Where,

C is the HSS torsional constant

F_{cr} shall be determined as follows:

For round HSS, F_{cr} shall be the larger of

$$F_{cr} = \frac{1.23E}{\sqrt{\frac{L}{D} \left(\frac{D}{t}\right)^{\frac{5}{4}}}} \quad (6.10.151a)$$

And,

$$F_{cr} = \frac{0.60E}{\left(\frac{D}{t}\right)^{\frac{3}{2}}} \quad (6.10.151b)$$

But, shall not exceed $0.6F_y$,

Where,

L = length of the member, mm

D = outside diameter, mm

For rectangular *HSS*

For $h/t \leq 2.45\sqrt{E/F_y}$

$$F_{cr} = 0.6F_y \quad (6.10.152)$$

For $2.45\sqrt{E/F_y} < h/t \leq 3.07\sqrt{E/F_y}$

$$F_{cr} = 0.6F_y \left(2.45\sqrt{E/F_y} \right) / (h/t) \quad (6.10.153)$$

For $3.07\sqrt{E/F_y} < h/t \leq 260$

$$F_{cr} = 0.458\pi^2 E / (h/t)^2 \quad (6.10.154)$$

10.8.3.2 HSS subject to combined torsion, shear, flexure and axial force

When the required torsional strength, T_r , is less than or equal to 20 percent of the available torsional strength, T_c , the interaction of torsion, shear, flexure and/or axial force for *HSS* shall be determined by Sec 10.8.1 and the torsional effects shall be neglected. When T_r exceeds 20 percent of T_c , the interaction of torsion, shear, flexure and/or axial force shall be limited by

$$\left(\frac{P_r}{P_c} + \frac{M_r}{M_c} \right) + \left(\frac{V_r}{V_c} + \frac{T_r}{T_c} \right)^2 \leq 1.0 \quad (6.10.155)$$

Where,

For design according to Sec 10.2.3.3 (LRFD)

P_r = required axial strength using LRFD load combinations, N

$P_c = \phi P_n$, design tensile or compressive strength in accordance with Sec 10.4 or 10.5, N

M_r = required flexural strength using LRFD load combinations, N-mm

$M_c = \phi_b M_n$, design flexural strength in accordance with Sec 10.6, N-mm

V_r =required shear strength using LRFD load combinations, N

$V_c = \phi_v V_n$, design shear strength in accordance with Sec 10.7, N

T_r =required torsional strength using LRFD load combinations, N-mm

$T_c = \phi_T T_n$, design torsional strength in accordance with Sec 10.8.3.1, N-mm

For design according to Sec 10.2.3.4 (ASD)

P_r = required axial strength using ASD load combinations, N

$P_c = P_n / \Omega$ allowable tensile or compressive strength in accordance with Sec 10.4 or 10.5, N

M_r =required flexural strength using ASD load combinations determined as per Sec10.2.5, N-mm

$M_c = M_n / \Omega_b$, allowable flexural strength in accordance with Sec 10.6, N-mm

V_r = required shear strength using ASD load combinations, N

$V_c = V_n / \Omega_v$, allowable shear strength in accordance with Sec 10.7, N

T_r = required torsional strength using ASD load combinations, N-mm

$T_c = T_n / \Omega_T$, allowable torsional strength in accordance with Sec 10.8.3.1, N-mm

10.8.3.3 Strength of non-HSS members under torsion and combined stress

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

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

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

$$F_n = F_y \quad (6.10.156)$$

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

$$F_n = 0.6F_y \quad (6.10.157)$$

- (c) For the limit state of buckling

$$F_n = F_{cr} \quad (6.10.158)$$

Where,

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

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

10.9 Evaluation of Existing Structures

This Section applies to the evaluation of the strength and stiffness under static vertical (gravity) loads of existing structures by structural analysis, by load tests, or by a combination of structural analysis and load tests when specified by the engineer of record or in the contract documents. For such evaluation, the steel grades are not limited to those listed in Sec 10.1.3.1. This Section does not address load testing for the effects of seismic loads or moving loads (vibrations).

10.9.1 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 (Sec 10.9.3), by load tests (Sec 10.9.4), or by a combination of structural analysis and load tests, as specified in the contract documents. Where load tests are used, the engineer of record shall first analyze the structure, prepare a testing plan, and develop a written procedure to prevent excessive permanent deformation or catastrophic collapse during testing.

10.9.2 Material Properties

Determination of Required Tests: The engineer of record shall determine the specific tests that are required from Sections 10.9.2.2 to 10.9.2.6 and specify the locations where they are required. Where available, the use of applicable project records shall be permitted to reduce or eliminate the need for testing.

10.9.2.1 Tensile properties

Tensile properties of members shall be considered in evaluation by structural analysis (Sec 10.9.3) or load tests (Sec 10.9.4). Such properties shall include the yield stress, tensile strength and percent elongation. Where available, certified mill test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM A6/A6M or A568/A568M, as applicable, shall be permitted for this purpose. Otherwise, tensile tests shall be conducted in accordance with ASTM A370 from samples cut from components of the structure.

10.9.2.2 Chemical composition

Where welding is anticipated for repair or modification of existing structures, the chemical composition of the steel shall be determined for use in preparing a welding procedure specification (WPS). Where available, results from certified mill test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM procedures shall be permitted for this purpose. Otherwise, analyses shall be conducted in accordance with ASTM A751 from the samples used to determine tensile properties, or from samples taken from the same locations.

10.9.2.3 Base metal notch toughness

Where welded tension splices in heavy shapes and plates as defined in Sec 10.1.3.1(d) are critical to the performance of the structure, the Charpy V-Notch toughness shall be determined in accordance with the provisions of Sec 10.1.3.1(d). If the notch toughness so determined does not meet the provisions of Sec 10.1.3.1(d), the engineer of record shall determine if remedial actions are required.

10.9.2.4 Weld metal

Where structural performance is dependent on existing welded connections, representative samples of weld metal shall be obtained. Chemical analysis and mechanical tests shall be made to characterize the weld metal. A determination shall be made of the magnitude and consequences of imperfections. If the requirements of AWS D1.1 are not met, the engineer of record shall determine if remedial actions are required.

10.9.2.5 Bolts and rivets

Representative samples of bolts shall be inspected to determine markings and classifications. Where bolts cannot be properly identified visually, representative samples shall be removed and tested to determine tensile strength in accordance with ASTM F606 or ASTM F606M and the bolt classified accordingly. Alternatively, the assumption that the bolts are ASTM A307 shall be permitted. Rivets shall be assumed to be ASTM A502, Grade 1, unless a higher grade is established through documentation or testing.

10.9.3 Evaluation by Structural Analysis**10.9.3.1 Dimensional data**

All dimensions used in the evaluation, such as spans, column heights, member spacings, bracing locations, cross-section dimensions, thicknesses and connection details, shall be determined from a field survey. Alternatively, when available, it shall be permitted to determine such dimensions from applicable project design or shop drawings with field verification of critical values.

10.9.3.2 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 Sec 10.2.2.

The available strength of members and connections shall be determined from applicable provisions of Sections 10.2 to 10.19 of this Specification.

10.9.3.3 Serviceability evaluation

Where required, the deformations at service loads shall be calculated and reported.

10.9.4 Evaluation by Load Tests**10.9.4.1 Determination of load rating by testing**

To determine the load rating of an existing floor or roof structure by testing, a test load shall be applied incrementally in accordance with the engineer of record's plan. The structure shall be visually inspected for signs of distress or imminent failure at each load level. Appropriate measures shall be taken if these or any other unusual conditions are encountered.

The tested strength of the structure shall be taken as the maximum applied test load plus the in-situ dead load. The live load rating of a floor structure shall be determined by setting the tested strength equal to $1.2 D + 1.6L$, where D is the nominal dead load and L is the nominal live load rating for the structure. The nominal live load rating of the floor structure shall not exceed that which can be calculated using applicable provisions of the specification. For roof structures, L_r , S , or R as defined in the Symbols, shall be substituted for L . More severe load combinations shall be used if required by specifications of Chapter 2 Part 6.

Periodic unloading shall be considered once the service load level is attained and after the onset of inelastic structural behavior is identified to document the amount of permanent set

and the magnitude of the inelastic deformations. Deformations of the structure, such as member deflections, shall be monitored at critical locations during the test, referenced to the initial position before loading. It shall be demonstrated, while maintaining maximum test load for one hour that the deformation of the structure does not increase by more than 10 percent above that at the beginning of the holding period. It is permissible to repeat the sequence if necessary to demonstrate compliance.

Deformations of the structure shall also be recorded 24 hours after the test loading is removed to determine the amount of permanent set. Because the amount of acceptable permanent deformation depends on the specific structure, no limit is specified for permanent deformation at maximum loading. Where it is not feasible to load test the entire structure, a segment or zone of not less than one complete bay, representative of the most critical conditions, shall be selected.

10.9.4.2 Serviceability evaluation

When load tests are prescribed, the structure shall be loaded incrementally to the service load level. Deformations shall be monitored for a period of one hour. The structure shall then be unloaded and the deformation recorded.

10.9.5 Evaluation Report

After the evaluation of an existing structure has been completed, the engineer of record shall prepare a report documenting the evaluation. The report shall indicate whether the evaluation was performed by structural analysis, by load testing or by a combination of structural analysis and load testing. Furthermore, when testing is performed, the report shall include the loads and load combination used and the load-deformation and time-deformation relationships observed. All relevant information obtained from design drawings, mill test reports and auxiliary material testing shall also be reported. Finally, the report shall indicate whether the structure, including all members and connections, is adequate to withstand the load effects.

10.10 Connections

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

10.10.1 General Provisions

10.10.1.1 Design basis

The design strength, ϕR_n and the allowable strength $\frac{R_n}{\Omega}$, of connections shall be determined in accordance with the provisions of this Section and the provisions of Sec 10.2.

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.

10.10.1.2 Simple connection

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

10.10.1.3 Moment connection

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 Sec 10.2.6.3.2.

10.10.1.4 Compression members with bearing joints

- (a) When columns bear on bearing plates or are finished to bear at splices, there shall be sufficient connectors to hold all parts securely in place.
- (b) When compression members other than columns are finished to bear, the splice material and its connectors shall be arranged to hold all parts in line and shall be proportioned for either (i) or (ii) below. It is permissible to use the less severe of the two conditions:
 - (i) An axial tensile force of 50 percent of the required compressive strength of the member; or
 - (ii) The moment and shear resulting from a transverse load equal to 2 percent of the required compressive strength of the member. The transverse load shall be applied at the location of the splice exclusive of other loads that act on the member. The member shall be taken as pinned for the determination of the shears and moments at the splice.

10.10.1.5 Splices in heavy sections

When tensile forces due to applied tension or flexure are to be transmitted through splices in heavy sections, as defined in Sections 10.1.3.1(c) and 10.1.3.1(d), by complete-joint-penetration groove (CJP) welds, material notch-toughness requirements as given in

Sections 10.1.3.1(c) and 10.1.3.1(d), weld access hole details as given in Sec 10.10.1.6 and thermal cut surface preparation and inspection requirements as given in Sec 10.1.3.2.2 shall apply. The foregoing provision is not applicable to splices of elements of built-up shapes that are welded prior to assembling the shape.

10.10.1.6 Beam copes and weld access holes

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

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

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

For heavy sections as defined in Sections 10.1.3.1(c) and 10.1.3.1(d), the thermally cut surfaces of beam copes and weld access holes shall be ground to bright metal and inspected by either magnetic particle or dye penetrant methods prior to deposition of splice welds. If the curved transition portion of weld access holes and beam copes are formed by predrilled or sawed holes, that portion of the access hole or cope need not be ground. Weld access holes and beam copes in other shapes need not be ground nor inspected by dye penetrant or magnetic particle methods.

10.10.1.7 Placement of welds and bolts

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

10.10.1.8 Bolts in combination with welds

Bolts shall not be considered as sharing the load in combination with welds, except that shear connections with any grade of bolts permitted by Sec 10.1.3.3 installed in standard holes or short slots transverse to the direction of the load are permitted to be considered to share the load with longitudinally loaded fillet welds. In such connections the available strength of the bolts shall not be taken as greater than 50 percent of the available strength of bearing-type bolts in the connection.

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

10.10.1.9 High-strength bolts in combination with rivets

In both new work and alterations, in connections designed as slip-critical connections in accordance with the provisions of Sec 10.10.3, high-strength bolts are permitted to be considered as sharing the load with existing rivets.

10.10.1.10 Limitations on bolted and welded connections

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

Column splices in all multi-story structures over 38 m in height

Connections of all beams and girders to columns and any other beams and girders on which the bracing of columns is dependent in structures over 38 m in height

In all structures carrying cranes of over 50 kN capacity: roof truss splices and connections of trusses to columns, column splices, column bracing, knee braces, and crane supports

Connections for the support of machinery and other live loads that produce impact or reversal of load

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

10.10.2 Welds

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

Sec 10.10.1.6 in lieu of AWS D1.1 Section 5.17.1

Sec 10.10.2.2.1 in lieu of AWS D1.1 Section 2.3.2

Table 6.10.5 in lieu of AWS D1.1 Table 2.1

Table 6.10.8 in lieu of AWS D1.1 Table 2.3

Table 6.10.14 in lieu of AWS D1.1 Table 2.4

Sec 10.2.3.9 and Sec 10.17 in lieu of AWS D1.1 Section 2, Part C

Sec 10.13.2.2 in lieu of AWS D1.1 Sections 5.15.4.3 and 5.15.4.4

10.10.2.1 Groove welds**10.10.2.1.1 Effective area**

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

The effective throat thickness of a complete-joint-penetration (CJP) groove weld shall be the thickness of the thinner part joined. The effective throat thickness of a partial-joint-penetration (PJP) groove weld shall be as shown in Table 6.10.4.

The effective weld size for flare groove welds, when filled flush to the surface of a round bar, a 90° bend in a formed section, or rectangular HSS shall be as shown in Table 6.10.5, unless other effective throats are demonstrated by tests. The effective size of flare groove welds filled less than flush shall be as shown in Table 6.10.5, less the greatest perpendicular dimension measured from a line flush to the base metal surface to the weld surface.

Larger effective throat thicknesses than those in Table 6.10.5 are permitted, provided the fabricator can establish by qualification the consistent production of such larger effective throat thicknesses. Qualification shall consist of sectioning the weld normal to its axis, at mid-length and terminal ends. Such sectioning shall be made on a number of combinations of material sizes representative of the range to be used in the fabrication.

10.10.2.1.2 Limitations

The minimum effective throat thickness of a partial-joint-penetration groove weld shall not be less than the size required to transmit calculated forces nor the size shown in Table 6.10.6. Minimum weld size is determined by the thinner of the two parts joined.

10.10.2.2 Fillet welds

10.10.2.2.1 Effective area

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

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

10.10.2.2.2 Limitations

The minimum size of fillet welds shall be not less than the size required to transmit calculated forces nor the size as shown in Table 6.10.7. These provisions do not apply to fillet weld reinforcements of partial- or complete-joint-penetration groove welds.

Table 6.10.4: Effective Throat of Partial-Joint-Penetration Groove Welds

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

Welding Process	Welding Position F (flat), H (horiz.), V (vert.), OH (overhead)	Groove Type (AWS D1.1, Figure 3.3)	Effective Throat
Shielded Metal Arc (SMAW)	All	45° Bevel	Depth of Groove Minus 3 mm
Gas Metal Arc (GMAW) Flux Cored Arc (FCAW)	V, OH	45° Bevel	Depth of Groove Minus 3 mm

Table 6.10.5: Effective Weld Sizes of Flare Groove Welds

Welding Process	Flare Bevel Groove ^[a]	Flare V-Groove
GMAW and FCAW-G	$\frac{5}{8}R$	$\frac{3}{4}R$
SMAW and FCAW-S	$\frac{5}{16}R$	$\frac{5}{8}R$
SAW	$\frac{5}{16}R$	$\frac{1}{2}R$

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

Table 6.10.6: Minimum Effective Throat Thickness of Partial-Joint-Penetration Groove Welds

Material Thickness of Thinner Part Joined (mm)	Minimum Effective Throat Thickness, ^[a] (mm)
To 6 inclusive	3
Over 6 to 13	5
Over 13 to 19	6
Over 19 to 38	8
Over 38 to 57	10
Over 57 to 150	13
Over 150	16

^[a] See Table 6.10.4.

Table 6.10.7: Minimum Size of Fillet Welds

Material Thickness of Thinner Part Joined (mm)	Minimum Size of Fillet Weld,^[a] (mm)
To 6 inclusive	3
Over 6 to 13	5
Over 13 to 19	6
Over 19	8

[a] Leg dimension of fillet welds. Single pass welds must be used.

Note: See Sec 10.10.2.2.2 for maximum size of fillet welds.

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

Along edges of material less than 6 mm thick, not greater than the thickness of the material.

Along edges of material 6 mm or more in thickness, not greater than the thickness of the material minus 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 2 mm provided the weld size is clearly verifiable.

The minimum effective length of fillet welds designed on the basis of strength shall be not less than four times the nominal size, or else the size of the weld shall be considered not to exceed 1/4th of its effective length. If longitudinal fillet welds are used alone in end connections of flat-bar tension members, the length of each fillet weld shall be not less than the perpendicular distance between them. For the effect of longitudinal fillet weld length in end connections upon the effective area of the connected member, see Sec 10.4.3.3.

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

$$\beta = 1.2 - 0.002(L/w) \leq 1.0 \quad (6.10.159)$$

Where,

L = actual length of end-loaded weld, mm

w = weld leg size, mm

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

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

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

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

For lap joints in which one connected part extends beyond an edge of another connected part that is subject to calculated tensile stress, fillet welds shall terminate not less than the size of the weld from that edge.

For connections where flexibility of the outstanding elements is required, when end returns are used, the length of the return shall not exceed four times the nominal size of the weld nor half the width of the part.

Fillet welds joining transverse stiffeners to plate girder webs 19 mm thick or less shall end not less than four times nor more than six times the thickness of the web from the web toe of the web-to-flange welds, except where the ends of stiffeners are welded to the flange.

Fillet welds that occur on opposite sides of a common plane shall be interrupted at the corner common to both welds.

Fillet welds in holes or slots are permitted to be used to transmit shear in lap joints or to prevent the buckling or separation of lapped parts and to join components of built-up members. Such fillet welds may overlap, subject to the provisions of Sec 10.10.2. Fillet welds in holes or slots are not to be considered plug or *slot welds*.

10.10.2.3 Plug and slot welds

10.10.2.3.1 Effective area

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

10.10.2.3.2 Limitations

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

The diameter of the holes for a plug weld shall not be less than the thickness of the part containing it plus 8 mm, rounded to the next larger odd even mm, nor greater than the minimum diameter plus 3 mm or $2\frac{1}{4}$ times the thickness of the weld.

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

The length of slot for a slot weld shall not exceed 10 times the thickness of the weld. The width of the slot shall be not less than the thickness of the part containing it plus 8 mm rounded to the next larger odd even mm, nor shall it be larger than $2\frac{1}{4}$ times the thickness of the weld. The ends of the slot shall be semicircular or shall have the corners rounded to a radius of not less than the thickness of the part containing it, except those ends which extend to the edge of the part.

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

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

10.10.2.4 Strength

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

Table 6.10.8: Available Strength of Welded Joints, N

Load Type and Direction Relative to Weld Axis	Pertinent Metal	ϕ and Ω	Nominal Strength (F_{BM} or F_W), N	Effective Area (A_{BM} or A_W), mm^2	Required Filler Metal Strength Level ^{[a][b]}
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COMPLETE-JOINT-PENETRATION GROOVE WELDS

Tension Normal to weld axis	Strength of the joint is controlled by the base metal			Matching filler metal shall be used. For T and corner joints with backing left in place, notch tough filler metal is required. Sec 10.10.2.6.	
Compression Normal to weld axis	Strength of the joint is controlled by the base metal			Filler metal with a strength level equal to or one strength level less than matching filler metal is permitted.	
Tension or Compression Parallel to weld axis	Tension or compression in parts joined parallel to a weld need not be considered in design of welds joining the parts.			Filler metal with a strength level equal to or less than matching filler metal is permitted.	
Shear	Strength of the joint is controlled by the base metal			Matching filler metal shall be used. ^[c]	

PARTIAL-JOINT-PENETRATION GROOVE WELDS INCLUDING FLARE VEE GROOVE AND FLARE BEVEL GROOVE WELDS

Tension Normal to weld axis	Base	$\phi = 0.90$ $\Omega = 1.67$	F_y	See 10.10.4	Filler metal with a strength level equal to or less than matching filler metal is permitted.
	Weld	$\phi = 0.80$ $\Omega = 1.88$	$0.60 F_{EXX}$	See 10.10.2.1.1	

Load Type and Direction Relative to Weld Axis	Pertinent Metal	ϕ and Ω	Nominal Strength (F_{BM} or F_W), N	Effective Area (A_{BM} or A_W), mm ²	Required Filler Metal Strength Level ^{[a][b]}
Compression Column to Base Plate and column splices designed per 10.10.1.4(a)					
Compression Connections of members designed to bear other than columns as described in 10.10.1.4(b)	Base	$\phi = 0.90$ $\Omega = 1.67$	F_y	See 10.10.4	
	Weld	$\phi = 0.80$ $\Omega = 1.88$	$0.60 F_{EXX}$	See 10.10.2.1.1	
Compression Connections not finished-to-bear	Base	$\phi = 0.90$ $\Omega = 1.67$	F_y	See 10.10.4	
	Weld	$\phi = 0.80$ $\Omega = 1.88$	$0.90 F_{EXX}$	See 10.10.2.1.1	
Tension or Compression Parallel to weld axis					
Shear	Base		Governed by 10.10.4		
	Weld	$\phi = 0.75$ $\Omega = 2.00$	$0.60 F_{EXX}$	See 10.10.2.1.1	

Filler metal with a strength level equal to or less than matching filler metal is permitted.

Load Type and Direction Relative to Weld Axis	Pertinent Metal	ϕ and Ω	Nominal Strength (F_{BM} or F_W), N	Effective Area (A_{BM} or A_W), mm ²	Required Filler Metal Strength Level ^{[a][b]}
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FILLET WELDS INCLUDING FILLETS IN HOLES AND SLOTS AND SKEWED T-JOINTS

Shear	Base	Governed by 10.10.4			Filler metal with a strength level equal to or less than matching filler metal is permitted.
	Weld	$\phi = 0.75$ $\Omega = 2.00$	$0.60F_{EXX}^{[d]}$	See 10.10.2.2.1	
Tension or Compression Parallel to weld axis	Tension or compression in parts joined parallel to a weld need not be considered in design of welds joining the parts.				

PLUG AND SLOT WELDS

Shear Parallel to faying surface on the effective area	Base	Governed by 10.10.4			Filler metal with a strength level equal to or less than matching filler metal is permitted.
	Weld	$\phi = 0.75$ $\Omega = 2.00$	$0.60F_{EXX}^{[d]}$	10.10.2.3.1	

^[a] For matching weld metal see AWS D1.1, Section 3.3.

^[b] Filler metal with a strength level one strength level greater than matching is permitted.

^[c] Filler metals with a strength level less than matching may be used for groove welds between the webs and flanges of built-up sections transferring shear loads, or in applications where high restraint is a concern. In these applications, the weld joint shall be detailed and the weld shall be designed using the thickness of the material as the effective throat, $\phi = 0.80, \Omega = 1.88$ and $0.60 F_{EXX}$ as the nominal strength.

^[d] Alternatively, the provisions of Sec 10.10.2.4 (a) are permitted provided the deformation compatibility of the various weld elements is considered. Alternatively, Sections 10.10.2.4 (b) and (c) are special applications of Sec 10.10.2.4 (a) that provide for deformation compatibility.

For the base metal

$$R_n = F_{BM} A_{BM} \quad (6.10.160)$$

For the weld metal

$$R_n = F_w A_w \quad (6.10.161)$$

Where,

F_{BM} = nominal strength of the base metal per unit area, MPa

F_w = nominal strength of the weld metal per unit area, MPa

A_{BM} = cross-sectional area of the base metal, mm²

A_w = effective area of the weld, mm²

The values of ϕ , Ω , F_{BM} , and F_w and limitations thereon are given in Table 6.10.8.

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

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

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

$$R_n = F_w A_w \quad (6.10.162)$$

Where,

$$F_w = 0.60 F_{EXX} (1.0 + 0.50 \sin^{1.5} \theta) \quad (6.10.163)$$

And,

F_{EXX} = electrode classification number, MPa

θ = angle of loading measured from the weld longitudinal axis, degrees

A_w = effective area of the weld, mm²

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

$$R_{nx} = \sum F_{wix} A_{wi} \quad R_{ny} = \sum F_{wiy} A_{wi} \quad (6.10.164)$$

Where,

A_{wi} = effective area of weld throat of any i^{th} weld element, mm^2

$$F_{wi} = 0.60 F_{EXX} (1.0 + 0.50 \sin^{1.5} \theta) f(p) \quad (6.10.165)$$

$$f(p) = [p(1.9 - 0.9p)]^{0.3} \quad (6.10.166)$$

F_{wi} = nominal stress in any i^{th} weld element, MPa

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

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

$\rho = \frac{\Delta_i}{\Delta_m}$, ratio of element i deformation to its deformation at maximum stress

w = weld leg size, mm

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

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

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

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

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

$$R_n = R_{wl} + R_{wt} \quad (6.10.167a)$$

Or,

$$R_n = 0.85R_{wl} + 1.5R_{wt} \quad (6.10.167b)$$

Where,

R_{wl} = the total nominal strength of longitudinally loaded fillet welds, as determined in accordance with Table 6.10.8, N

R_{wt} = the total nominal strength of transversely loaded fillet welds, as determined in accordance with Table 6.10.8 without the alternate in Sec 10.10.2.4(a), N

10.10.2.5 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.

10.10.2.6 Filler metal requirements

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

Filler metal with a specified Charpy V-Notch (CVN) toughness of 27.12 N-m (27 Joule) at 4°C shall be used in the following joints:

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

Complete-joint-penetration groove welded splices subject to tension normal to the effective area in heavy sections as defined in Sections 10.1.3.1(c) and 10.1.3.1(d).

10.10.2.7 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.

10.10.3 Bolts and Threaded Parts

10.10.3.1 High-strength bolts

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

When assembled, all joint surfaces, including those adjacent to the washers, shall be free of scale, except tight mill scale. All ASTM A325/A325M and A490/A490M bolts shall be tightened to a bolt tension not less than given in Table 6.10.9, except as noted below. Except as permitted below, installation shall be assured by any of the following methods: turn-of-nut method, a direct tension indicator, calibrated wrench or alternative design bolt.

- (a) Bolts are permitted to be installed to only the snug-tight condition when used in bearing-type connections.
- (b) Tension or combined shear and tension applications, for ASTM A325 or A325M bolts only, where loosening or fatigue due to vibration or load fluctuations are not design considerations.

Table 6.10.9: Minimum Bolt Pretension, kN*

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

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

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

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

In slip-critical connections in which the direction of loading is toward an edge of a connected part, adequate available bearing strength shall be provided based upon the applicable requirements of Sec 10.10.3.10.

When bolt requirements cannot be provided by ASTM A325/A325M, F1852, or A490/A490M bolts because of requirements for lengths exceeding 12 diameters or diameters exceeding 38 mm, bolts or threaded rods conforming to ASTM A354 Grade BC, A354 Grade BD, or A449 are permitted to be used in accordance with the provisions for threaded rods in Table 6.10.10.

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

10.10.3.2 Size and use of holes

The maximum sizes of holes for bolts are given in Table 6.10.11, except that larger holes, required for tolerance on location of anchor rods in concrete foundations, are permitted in column base details.

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

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

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

Long-slotted holes are permitted in only one of the connected parts of either a slip-critical or bearing-type connection at an individual faying surface. Long-slotted holes are permitted without regard to direction of loading in slip-critical connections, but shall be normal to the direction of load in bearing-type connections. Where long-slotted holes are

used in an outer ply, plate washers, or a continuous bar with standard holes, having a size sufficient to completely cover the slot after installation, shall be provided. In high-strength bolted connections, such plate washers or continuous bars shall be not less than 8 mm thick and shall be of structural grade material, but need not be hardened. If hardened washers are required for use of high-strength bolts, the hardened washers shall be placed over the outer surface of the plate washer or bar.

Table 6.10.10: Nominal Stress of Fasteners and Threaded Parts, MPa

Description of Fasteners	Nominal Tensile Stress, F_{nt} , MPa	Nominal Shear Stress in Bearing-Type Connections, F_{nv} , MPa
A307 bolts	310 [a][b]	165 [b] [c] [f]
A325/A325M bolts, when threads are not excluded from shear planes	620 [e]	330 [f]
A325/A325M bolts, when threads are excluded from shear planes	620 [e]	414 [f]
A490/A490M bolts, when threads are not excluded from shear planes	780 [e]	414 [f]
A490/A490M bolts, when threads are excluded from shear planes	780 [e]	520 [f]
Threaded parts meeting the requirements of Sec 10.1.3.4, when threads are not excluded from shear planes	$0.75 F_u$ [a][d]	$0.40 F_u$
Threaded parts meeting the requirements of Sec 10.1.3.4, when threads are excluded from shear planes	$0.75 F_u$ [a][d]	$0.50 F_u$

[a] Subject to the requirements of Sec 10.17.

[b] For A307 bolts the tabulated values shall be reduced by 1 percent for each 2 mm over 5 diameters of length in the grip.

[c] Threads permitted in shear planes.

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

[e] For A325 or A325M and A490 or A490M bolts subject to tensile fatigue loading, see Sec 10.2.3.9

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

Table 6.10.11: Nominal Hole Dimensions, mm

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

[a] Clearance provided allows the use of a 25 mm. bolt if desirable.

10.10.3.3 Minimum spacing

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

10.10.3.4 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 6.10.12, or as required in Sec 10.10.3.10. 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 6.10.13.

Table 6.10.12: Minimum Edge Distance,^[a] mm, from Center of Standard Hole^[b] to Edge of Connected Part

Bolt Diameter (mm)	At Sheared Edges	At Rolled Edges of
		Plates, Shapes or Bars, or Thermally Cut Edges ^[c]
16	28	22
20	34	26
22	38 ^[d]	28
24	42 ^[d]	30
27	48	34

Bolt Diameter (mm)	At Sheared Edges	At Rolled Edges of Plates, Shapes or Bars, or Thermally Cut Edges ^[c]
30	52	38
36	64	46
Over 36	1.75d	1.25d

Notes: Table 6.10.12

^[a] Lesser edge distances are permitted to be used provided provisions of Sec 10.10.3.10, as appropriate, are satisfied.

^[b] For oversized or slotted holes, see Table 6.10.13.

^[c] All edge distances in this column are permitted to be reduced 3 mm when the hole is at a point where required strength does not exceed 25 percent of the maximum strength in the element.

^[d] These are permitted to be 32 mm at the ends of beam connection angles and shear end plates.

Table 6.10.13: Values of Edge Distance Increment C₂, mm

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

^[a] When length of slot is less than maximum allowable (see Table 6.10.11). C₂ is permitted to be reduced by one-half the difference between the maximum and actual slot lengths.

10.10.3.5 Maximum spacing and edge distance

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

- For painted members or unpainted members not subject to corrosion, the spacing shall not exceed 24 times the thickness of the thinner plate or 305 mm.
- For unpainted members of weathering steel subject to atmospheric corrosion, the spacing shall not exceed 14 times the thickness of the thinner plate or 180 mm.

10.10.3.6 Tension and shear strength of bolts and threaded parts

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

$$R_n = F_n A_b \quad (6.10.168)$$

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

Where,

F_n = nominal tensile stress F_{nt} , or shear stress, F_{nv} from Table 6.10.10, MPa

A_b = nominal unthreaded body area of bolt or threaded part (for upset rods, see footnote d, Table 6.10.10, mm²)

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

10.10.3.7 Combined tension and shear in bearing-type connection

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

$$R_n = F'_{nt} A_b \quad (6.10.169)$$

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

Where,

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

$$F'_{nt} = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v \leq F_{nt} \text{ (LRFD)} \quad (6.10.170a)$$

$$F'_{nt} = 1.3F_{nt} - \frac{\Omega F_{nt}}{\phi F_{nv}} f_v \leq F_{nt} \text{ (ASD)} \quad (6.10.170b)$$

F_{nt} = nominal tensile stress from Table 6.10.10, MPa

F_{nv} = nominal shear stress from Table 6.10.10, MPa

f_v = the required shear stress, MPa

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

10.10.3.8 High-strength bolts in slip-critical connections

High-strength bolts in slip-critical connections are permitted to be designed to prevent slip either as a serviceability limit state or at the required strength limit state. The connection must also be checked for shear strength in accordance with Sections 10.10.3.6 and 10.10.3.7 and bearing strength in accordance with Sections 10.10.3.1 and 10.10.3.10.

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

The design slip resistance, ϕR_n and the allowable slip resistance, $\frac{R_n}{\Omega}$ shall be determined for the limit state of slip as follows:

$$R_n = \mu D_u h_{sc} T_b N_s \quad (6.10.171)$$

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

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

For connections designed to prevent slip at the required strength level

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

Where,

μ = mean slip coefficient for Class A or B surfaces, as applicable, or as established by tests

= 0.35 for Class A surfaces (unpainted clean mill scale steel surfaces or surfaces with Class A coatings on blast-cleaned steel and hot-dipped galvanized and roughened surfaces)

= 0.50 for Class B surfaces (unpainted blast-cleaned steel surfaces or surfaces with Class B coatings on blast-cleaned steel)

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

h_{sc} = hole factor determined as follows:

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

N_s = number of slip planes

T_b = minimum fastener tension given in Table 6.10.9, kN

10.10.3.9 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 Sec 10.10.3.8, shall be multiplied by the factor, k_s , as follows:

$$k_s = 1 - \frac{T_u}{D_u T_b N_b} \quad (\text{LRFD}) \quad (6.10.172a)$$

$$k_s = 1 - \frac{1.5T_a}{D_u T_b N_b} \quad (\text{ASD}) \quad (6.10.172b)$$

Where,

N_b = number of bolts carrying the applied tension

T_a = tension force due to ASD load combinations, kN

T_b = minimum fastener tension given in Table 6.10.9, kN

T_u = tension force due to LRFD load combinations, kN

10.10.3.10 Bearing strength at bolt holes

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

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

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:

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

$$R_n = 1.2L_c t F_u \leq 2.4dt F_u \quad (6.10.173a)$$

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

$$R_n = 1.5L_c t F_u \leq 3.0dt F_u \quad (6.10.173b)$$

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

$$R_n = 1.0L_c t F_u \leq 2.0dt F_u \quad (6.10.173c)$$

For connections made using bolts that pass completely through an unstiffened box member or *HSS* see Sec 10.10.7 and Eq. 6.10.180,

Where,

d = nominal bolt diameter, mm

F_u = specified minimum tensile strength of the connected material, MPa

L_c = clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material, mm

t = thickness of connected material, mm

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

Bearing strength shall be checked for both bearing-type and slip-critical connections. The use of oversized holes and short- and long-slotted holes parallel to the line of force is restricted to slip-critical connections per Sec 10.10.3.2.

10.10.3.11 Special fasteners

The nominal strength of special fasteners other than the bolts presented in Table 6.10.10 shall be verified by tests.

10.10.3.12 Tension fasteners

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

10.10.4 Affected Elements of Members and Connecting Elements

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

10.10.4.1 Strength of elements in tension

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

For tensile yielding of connecting elements:

$$R_n = F_y A_g \quad (6.10.174)$$

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

For tensile rupture of connecting elements:

$$R_n = F_u A_e \quad (6.10.175)$$

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

Where,

A_e = effective net area as defined in Sec 10.4.3.3, mm²; for bolted splice plates, $A_e = A_n \leq 0.85 A_g$

10.10.4.2 Strength of elements in shear

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

For shear yielding of the element:

$$R_n = 0.60 F_y A_g \quad (6.10.176)$$

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

For shear rupture of the element:

$$R_n = 0.60F_u A_{nv} \quad (6.10.177)$$

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

Where,

$$A_{nv} = \text{net area subject to shear, mm}^2$$

10.10.4.3 Block shear strength

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

$$R_n = 0.60F_u A_{nv} + U_{bs}F_u A_{nt} \leq 0.6F_y A_{gv} + U_{bs}F_u A_{nt} \quad (6.10.178)$$

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

Where,

$$A_{gv} = \text{gross area subject to shear, mm}^2$$

$$A_{nt} = \text{net area subject to tension, mm}^2$$

$$A_{nv} = \text{net area subject to shear, mm}^2$$

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

10.10.4.4 Strength of elements in compression

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

For $KL/r \leq 25$

$$P_n = F_y A_g \quad (6.10.179)$$

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

For $KL/r > 25$ the provisions of Sec 10.5 apply.

10.10.5 Fillers

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

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

For fillers that are equal to or less than 19 mm thick, the shear strength of the bolts shall be multiplied by the factor $[1 - 0.0154(t - 6)]$, where t is the total thickness of the fillers up to 19 mm;

The fillers shall be extended beyond the joint and the filler extension shall be secured with enough bolts to uniformly distribute the total force in the connected element over the combined cross section of the connected element and the fillers;

The size of the joint shall be increased to accommodate a number of bolts that is equivalent to the total number required in (2) above; or

The joint shall be designed to prevent slip at required strength levels in accordance with Sec 10.10.3.8.

10.10.6 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.

10.10.7 Bearing Strength

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

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

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

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

$$R_n = 1.8F_y A_{pb} \quad (6.10.180)$$

Where,

F_y = specified minimum yield stress, MPa

A_{pb} = projected bearing area, mm²

For expansion rollers and rockers:

If $d \leq 635$ mm

$$R_n = 1.2(F_y - 90)ld/20 \quad (6.10.181)$$

If $d > 635$ mm

$$R_n = 30.2(F_y - 90)l\sqrt{d}/20 \quad (6.10.182)$$

Where,

d = diameter, mm

l = length of bearing, mm

10.10.8 Column Bases and Bearing on Concrete

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

In the absence of the Code regulations, the design bearing strength, $\phi_c P_n$ and the allowable bearing strength, $\frac{P_n}{\Omega_c}$ for the limit state of concrete crushing are permitted to be taken as follows:

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

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

On the full area of a concrete support:

$$P_p = 0.8f'_c A_1 \quad (6.10.183)$$

On less than the full area of a concrete support:

$$P_p = 0.8f'_c A_1 \sqrt{A_2/A_1} \leq 1.7f'_c A_1 \quad (6.10.184)$$

Where,

A_1 = area of steel concentrically bearing on a concrete support, mm²

A_2 = maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, mm²

10.10.9 Anchor Rods and Embedments

Anchor rods shall be designed to provide the required resistance to loads on the completed structure at the base of columns including the net tensile components of any bending moment that may result from load combinations stipulated in Sec 10.2.2. The anchor rods shall be designed in accordance with the requirements for threaded parts in Table 6.10.10.

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

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

10.10.10 Flanges and Webs with Concentrated Forces

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

When the required strength exceeds the available strength as determined for the limit states listed in this Section, stiffeners and/or doublers shall be provided and shall be sized for the difference between the required strength and the available strength for the applicable limit state. Stiffeners shall also meet the design requirements in Sec 10.10.10.8. Doublers shall also meet the design requirement in Sec 10.10.10.9.

Stiffeners are required at unframed ends of beams in accordance with the requirements of Sec 10.10.10.7.

10.10.10.1 Flange local bending

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

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

$$R_n = 6.25t_f^2 F_{yf} \quad (6.10.185)$$

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

Where,

F_{yf} = specified minimum yield stress of the flange, MPa

t_f = thickness of the loaded flange, mm

If the length of loading across the member flange is less than $0.15b_f$, where b_f is the member flange width, Eq. 6.10.185 need not be checked.

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

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

10.10.10.2 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:

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

$$R_n = (5k + N)F_{yw}t_w \quad (6.10.186)$$

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

$$R_n = (2.5k + N)F_{yw}t_w \quad (6.10.187)$$

Where,

k = distance from outer face of the flange to the web toe of the fillet, mm

F_{yw} = specified minimum yield stress of the web, MPa

N = length of bearing (not less than k for end beam reactions), mm

t_w = web thickness, mm

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

10.10.10.3 Web Crippling

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

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

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

The nominal strength, R_n shall be determined as follows:

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

$$R_n = 0.80t_w^2 \left[1 + 3 \left(\frac{N}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{EF_{yw}t_f}{t_w}} \quad (6.10.188)$$

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

For $N/d \leq 0.2$

$$R_n = 0.40t_w^2 \left[1 + 3 \left(\frac{N}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{EF_{yw}t_f}{t_w}} \quad (6.10.189a)$$

For $N/d > 0.2$

$$R_n = 0.40t_w^2 \left[1 + \left(\frac{4N}{d} - 0.2 \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{EF_{yw}t_f}{t_w}} \quad (6.10.189b)$$

Where,

d = overall depth of the member, mm

t_f = flange thickness, mm

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

10.10.10.4 Web sidesway buckling

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

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

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

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

If the compression flange is restrained against rotation:

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

$$R_n = \frac{C_r t_w^3 t_f}{h^2} \left[1 + 0.4 \left(\frac{h/t_w}{l/b_f} \right)^3 \right] \quad (6.10.190)$$

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

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

If the compression flange is not restrained against rotation

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

$$R_n = \frac{C_r t_w^3 t_f}{h^2} \left[0.4 \left(\frac{h/t_w}{l/b_f} \right)^3 \right] \quad (6.10.191)$$

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

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

In Equations 6.10.190 and 6.10.191, the following definitions apply:

b_f = flange width, mm

C_r = 6.62×10^6 MPa when $M_u < M_y$ (LRFD) or $1.5 M_a < M_y$ (ASD) at the location of the force

= 3.31×10^6 MPa when $M_u \geq M_y$ (LRFD) or $1.5 M_a \geq M_y$ (ASD) at the location of the force

h = clear distance between flanges less fillet or corner radius for rolled shapes;
distance between adjacent lines of fasteners or clear distance between flanges
when welds are used for built-up shapes, mm

l = largest laterally unbraced length along either flange at the point of load, mm

t_f = flange thickness, mm

t_w = web thickness, mm

10.10.10.5 Web compression buckling

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

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

$$R_n = \frac{24t_w^3 \sqrt{EF_{yw}}}{h} \quad (6.10.192)$$

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

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

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

10.10.10.6 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 web panel zone for limit state of shear yielding shall be determined as follows:

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

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

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

For $P_r \leq 0.4P_c$

$$R_n = 0.60F_y d_c t_w \quad (6.10.193)$$

For $P_r > 0.4P_c$

$$R_n = 0.60F_y d_c t_w \left(1.4 - \frac{P_r}{P_c}\right) \quad (6.10.194)$$

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

For $P_r \leq 0.75P_c$

$$R_n = 0.60F_y d_c t_w \left(1 + \frac{3b_{cf} t_{cf}^2}{d_b d_c t_w} \right) \quad (6.10.195)$$

For $P_r > 0.75P_c$

$$R_n = 0.60F_y d_c t_w \left(1 + \frac{3b_{cf} t_{cf}^2}{d_b d_c t_w} \right) \left(1.9 - \frac{1.2P_r}{P_c} \right) \quad (6.10.196)$$

In Equations 6.10.193 to 6.10.196, the following definitions apply:

A = column cross-sectional area, mm²

b_{cf} = width of column flange, mm

d_b = beam depth, mm

d_c = column depth, mm

F_y = specified minimum yield stress of the column web, MPa

P_c = P_y , N (LRFD)

P_c = 0.6 P_y , N (ASD)

P_r = required strength, N

P_y = $F_y A$, axial yield strength of the column, N

t_{cf} = thickness of the column flange, mm

t_w = column web thickness, mm

When required, doubler plate(s) or a pair of diagonal stiffeners shall be provided within the boundaries of the rigid connection whose webs lie in a common plane. See Sec 10.10.10.9 for doubler plate design requirements.

10.10.10.7 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.

10.10.10.8 Additional stiffeners requirements for concentrated forces

Stiffeners required to resist tensile concentrated forces shall be designed in accordance with the requirements of Sec 10.4 and welded to the loaded flange and the web. The welds to the flange shall be sized for the difference between the required strength and available limit state strength. The stiffener to web welds shall be sized to transfer to the web the algebraic difference in tensile force at the ends of the stiffener.

Stiffeners required to resist compressive concentrated forces shall be designed in accordance with the requirements in Sections 10.5.6.2 and 10.10.4.4 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 Sec 10.10.7.

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) as per the requirements of Sections 10.5.6.2 and 10.10.4.4.

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

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

The width of each stiffener plus one-half the thickness of the column web shall not be less than one-third of the width of the flange or moment connection plate delivering the concentrated force.

- (a) Thickness of a stiffener shall not be less than one-half the thickness of the flange or moment connection plate delivering the concentrated load, and greater than or equal to the width divided by 15.

- (b) Transverse stiffeners shall extend a minimum of one-half the depth of the member except as required in Sec. 10.10.10.5 and 10.10.10.7.

10.10.10.9 Additional doubler plate requirements for concentrated forces

Doubler plates required for compression strength shall be designed according to the requirements of Sec 10.5.

Doubler plates required for tensile strength shall be designed in accordance with the requirements of Sec 10.4.

Doubler plates required for shear strength (Sec 10.10.10.6) shall be designed following the provisions of Sec 10.7.

In addition, doubler plates shall comply with the following criteria:

- (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 total force transmitted to doubler plate.

10.11 Design of Hss and Box Member Connections

This Section covers member strength design considerations pertaining to connections to HSS members and box sections of uniform wall thickness. See also Section 10.10 for additional requirements for bolting to HSS.

10.11.1 Concentrated Forces on HSS

10.11.1.1 Notation and definitions

The notation and their definitions related to this Section are given in Sec 10.1.2.

10.11.1.2 Limits of applicability

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

For HSS:

$$\text{Strength: } F_y \leq 360 \text{ MPa}$$

$$\text{Ductility: } \frac{F_y}{F_u} \leq 0.8$$

Other limits apply for specific criteria.

10.11.1.3 Concentrated force distributed transversely

10.11.1.3.1 Criterion for round HSS

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

$$R_n = F_y t^2 [5.5/(1 - 0.81 B_p/D)] Q_f \quad (6.10.197)$$

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

Where, the Equations for Q_f is given in Sec 10.11.2.2 (Eq. 6.10.209).

Additional limits of applicability are:

$$0.2 < B_p/D \leq 1.0.$$

$$\text{For T-connections: } D/t \leq 50$$

$$\text{For cross-connections: } D/t \leq 40$$

10.11.1.3.2 Criterion for rectangular HSS

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

Additional limits of applicability are:

$$0.25 < B_p/B \leq 1.0$$

$$\text{For loaded HSS wall } B/t \leq 35$$

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

$$R_n = \left[\frac{10}{\frac{F_y t}{\left(\frac{B}{t}\right)}} \right] B_P \leq F_{yP} t_P B_P \quad (6.10.198)$$

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

For the limit state of shear yielding (punching),

$$R_n = 0.6 F_y t [2t_P + 2B_{ep}] \quad (6.10.199)$$

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

Where, $B_{ep} = 10B_P / (B/t) \leq B_P$

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

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

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

For the limit state of sidewall local yielding,

$$R_n = 2F_y t [5k + N] \quad (6.10.200)$$

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

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

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

$$R_n = 1.6t^2 \left[1 + \frac{3N}{(H-3t)} \right] (EF_y)^{0.5} Q_f \quad (6.10.201)$$

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

Where, the Equations for Q_f is given in Sec 10.11.2.3 (Eq. 6.10.218).

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

$$R_n = \left[\frac{48t^3}{(H-3t)} \right] (EF_y)^{0.5} Q_f \quad (6.10.202)$$

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

Where, the Equations for Q_f is given in Sec 10.11.2.3 (Eq. 6.10.218).

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

$$L_e = 2 \left[\frac{10}{\left(\frac{B}{t}\right)} \right] \left[\frac{(F_t t)}{(F_{yp} t_p)} \right] B_p \leq 2B_p \quad (6.10.203)$$

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

In lieu of Eq. 6.10.203, this requirement may be satisfied by other rational approaches.

10.11.1.4 Concentrated Force distributed longitudinally at the center of the HSS diameter or width and acting perpendicular to the HSS Axis

When a concentrated force is distributed longitudinally along the axis of the HSS at the center of the HSS diameter or width, and also acts perpendicular to the axis direction of the HSS (or has a component perpendicular to the axis direction of the HSS), the design strength, ϕR_n and the allowable strength, R_n/Ω , perpendicular to the HSS axis shall be determined for the limit state of chord plastification as follows.

10.11.1.4.1 Criterion for round HSS

An additional limit of applicability is:

$D/t \leq 50$ for T-connections and $D/t \leq 40$ for cross-connections

$$R_n = 5.5F_y t^2 \left(1 + \frac{0.25N}{D} \right) Q_f \quad (6.10.204)$$

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

Where, the Equation for Q_f is given in Sec 10.11.2.2 (Eq. 6.10.209).

10.11.1.4.2 Criterion for rectangular HSS

An additional limit of applicability is:

B/t for the loaded HSS wall ≤ 40

$$R_n = \left[\frac{F_y t^2}{(1 - t_p/B)} \right] \left[\frac{2N}{B} + 4 \left(1 - \frac{t_p}{B} \right)^{0.5} Q_f \right] \quad (6.10.205)$$

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

Where, $Q_f = (1 - U^2)^{0.5}$

The Equation for U is given in Sec 10.11.2.3 (Eq. 6.10.220).

10.11.1.5 Concentrated force distributed longitudinally at center of HSS width and acting parallel to HSS axis

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

$$F_{yp} t_p \leq F_u t \quad (6.10.206)$$

10.11.1.6 Concentrated axial force on the end of a rectangular HSS with a cap plate

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

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

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

For the limit state of wall local yielding, for one wall,

$$R_n = F_y t [5t_p + N] \leq B F_y t \quad (6.10.207)$$

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

For the limit state of wall local crippling, for one wall,

$$R_n = 0.8t^2 \left[1 + \left(\frac{6N}{B} \right) \left(\frac{t}{t_p} \right)^{1.5} \right] \left[\frac{EF_y t_p}{t} \right]^{0.5} \quad (6.10.208)$$

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

10.11.2 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:

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

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

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

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

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

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

10.11.2.1 Notation and Definitions

The notation and their definitions related to this Section are given in Sec 10.1.2.

10.11.2.2 Criteria for round HSS

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

When the chord is in tension,

$$Q_f = 1$$

When the chord is in compression,

$$Q_f = 1.0 - 0.3U(1 + U) \quad (6.10.209)$$

Where, U is the utilization ratio given by

$$U = \left| \frac{P_r}{(A_g F_c)} + \frac{M_r}{(S F_c)} \right| \quad (6.10.210)$$

And,

P_r = required axial strength in chord, N; for K-connections, P_r is to be determined on the side of the joint that has the lower compression stress (lower U)

M_r = required flexural strength in chord, N-mm.

A_g = chord gross area, mm²

F_c = available stress, MPa.

S = chord elastic section modulus, mm³

For design according to Section 10.2.3.3 (LRFD):

$P_r = P_u$ = required axial strength in chord, using LRFD load combinations, N

$M_r = M_u$ = required flexural strength in chord, using LRFD load combinations, N-mm.

$F_c = F_y$, MPa.

For design according to Section 10.2.3.4 (ASD):

$P_r = P_a$ = required axial strength in chord, using ASD load combinations, N

$M_r = M_a$ = required flexural strength in chord, using ASD load combinations, N-mm.

$f_c = 0.6f_y$, MPa.

10.11.2.2.1 Limits of applicability

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

- (a) Joint eccentricity: $-0.55D \leq e \leq 0.25D$, where D is the chord diameter and e is positive away from the branches
- (b) Branch angle: $\theta \geq 30^\circ$
- (c) Chord wall slenderness: ratio of diameter to wall thickness less than or equal to 50 for T-, Y- and K-connections; less than or equal to 40 for cross-connections
- (d) Tension branch wall slenderness: ratio of diameter to wall thickness less than or equal to 50
- (e) Compression branch wall slenderness: ratio of diameter to wall thickness $\leq 0.05E/F_y$.
- (f) Width ratio: $0.2 \leq \frac{D_b}{D} \leq 1.0$ in general, and $0.4 \leq \frac{D_b}{D} \leq 1.0$ for gapped K-connections
- (g) If a gap connection: g greater than or equal to the sum of the branch wall thicknesses
- (h) If an overlap connection: $25\% \leq O_v \leq 100\%$, where $O_v = \left(\frac{q}{p}\right) \times 100\%$. P is the projected length of the overlapping branch on the chord; q is the overlap length measured along the connecting face of the chord beneath the two branches. For overlap connections, the larger (or if equal diameter, the thicker) branch is a “thru member” connected directly to the chord.

- (i) Branch thickness ratio for overlap connections: thickness of overlapping branch to be less than or equal to the thickness of the overlapped branch.
- (j) Strength: $F_y \leq 360$ MPa. for chord and branches
- (k) Ductility: $F_y/F_u \leq 0.8$

10.11.2.2.2 Branches with axial loads in T-, Y- and cross-connections

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

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

$$P_n \sin \theta = F_y t^2 [3.1 + 15.6\beta^2] \gamma^{0.2} Q_f \quad (6.10.211)$$

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

For the limit state of shear yielding (punching),

$$P_n = 0.6 F_y t \pi D_b \left[\frac{(1 + \sin \theta)}{2 \sin^2 \theta} \right] \quad (6.10.212)$$

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

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

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

$$P_n \sin \theta = F_y t^2 \left[\frac{5.7}{(1 - 0.81\beta)} \right] Q_f \quad (6.10.213)$$

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

10.11.2.2.3 Branches with axial loads in K-connections

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

For the limit state of chord plastification,

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

For the compression branch:

$$P_n \sin \theta = F_y t^2 \left[2.0 + \frac{11.33 D_b}{D} \right] Q_g Q_f \quad (6.10.214)$$

Where, D_b refers to the compression branch only, and

$$Q_g = \gamma^{0.2} \left[1 + \frac{0.024 \gamma^{1.2}}{e^{\left(\frac{0.5g}{t} - 1.33\right)} + 1} \right] \quad (6.10.215)$$

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

For the tension branch,

$$P_n \sin \theta = (P_n \sin \theta)_{compression\ branch} \quad (6.10.216)$$

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

$$P_n = 0.6 F_y t \pi D_b \left[\frac{(1 + \sin \theta)}{2 \sin^2 \theta} \right] \quad (6.10.217)$$

$$\phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.58 \text{ (ASD)}$$

10.11.2.3 Criteria for rectangular HSS

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

When the chord is in tension,

$$Q_f = 1$$

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

$$Q_f = 1.3 - 0.4U/\beta \leq 1 \quad (6.10.218)$$

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

$$Q_f = 1.3 - 0.4U/\beta_{eff} \leq 1 \quad (6.10.219)$$

Where, U is the utilization ratio given by

$$U = \left| \frac{P_r}{(A_g F_c)} + \frac{M_r}{(S F_c)} \right| \quad (6.10.220)$$

And,

P_r = required axial strength in chord, N. For gapped K-connections, P_r is to be determined on the side of the joint that has the higher compression stress (higher U).

M_r = required flexural strength in chord, N-mm.

A_g = chord gross area, mm²

F_c = available stress, MPa.

S = chord elastic section modulus, mm³

For design according to Section 10.2.3.3 (LRFD):

$P_r = P_u$ = required axial strength in chord, using *LRFD load combinations*, N

$M_r = M_u$ = required flexural strength in chord, using LRFD load combinations, N-mm.

$F_c = F_u$, MPa.

For design according to Section 10.2.3.4 (ASD):

$P_r = P_a$ = required axial strength in chord, using *ASD load combinations*, N.

$M_r = M_a$ = required flexural strength in chord, using ASD load combinations, N-mm.

$F_c = 0.6F_y$, MPa.

10.11.2.3.1 Limits of applicability

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

- (a) Joint eccentricity: $-0.55H \leq e \leq 0.25H$, where H is the chord depth and e is positive away from the branches
- (b) Branch angle: $\theta \geq 30^\circ$

- (c) Chord wall slenderness: ratio of overall wall width to thickness less than or equal to 35 for gapped K-connections and T-, Y- and cross-connections; less than or equal to 30 for overlapped K-connections
- (d) Tension branch wall slenderness: ratio of overall wall width to thickness less than or equal to 35
- (e) Compression branch wall slenderness: ratio of overall wall width to thickness less than or equal to $1.25(E/F_{yb})^{0.5}$ and also less than 35 for gapped K-connections and T-, Y- and cross-connections; less than or equal to $1.1(E/F_{yb})^{0.5}$ for overlapped K-connections
- (f) Width ratio: ratio of overall wall width of branch to overall wall width of chord greater than or equal to 0.25 for T-, Y-, cross- and overlapped K-connections; greater than or equal to 0.35 for gapped K-connections
- (g) Aspect ratio: $0.5 \leq \text{ratio of depth to width} \leq 2.0$
- (h) Overlap: $25\% \leq O_v \leq 100\%$, where $O_v = \left(\frac{q}{p}\right) \times 100\%$. p is the projected length of the overlapping branch on the chord; q is the overlap length measured along the connecting face of the chord beneath the two branches. For overlap connections, the larger (or if equal width, the thicker) branch is a “thru member” connected directly to the chord
- (i) Branch width ratio for overlap connections: ratio of overall wall width of overlapping branch to overall wall width of overlapped branch greater than or equal to 0.75
- (j) Branch thickness ratio for overlap connections: thickness of overlapping branch to be less than or equal to the thickness of the overlapped branch
- (k) Strength: $F_y \leq 360 \text{ MPa}$. for chord and branches
- (l) Ductility: $F_y/F_u \leq 0.8$
- (m) Other limits apply for specific criteria

10.11.2.3.2 Branches with axial loads in T-,Y- and cross-connections

For T-, Y- and cross-connections, the design strength of the branch, ϕP_n or the allowable strength of the branch, P_n/Ω , shall be the lowest value obtained according to the limit states of chord wall plastification, shear yielding (punching), sidewall strength and local yielding due to uneven load distribution. In addition to the limits of applicability in Section 10.11.2.3a, β shall not be less than 0.25.

For the limit state of chord wall plastification,

$$P_n \sin \theta = F_y t^2 \left[\frac{2\eta}{(1-\beta)} + \frac{4}{(1-\beta)^{0.5}} \right] Q_f \quad (6.10.221)$$

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

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

For the limit state of shear yielding (punching),

$$P_n \sin \theta = 0.6 F_y t B [2\eta + 2\beta_{eop}] \quad (6.10.222)$$

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

In Eq. 6.10.222, the effective outside punching parameter $\beta_{eop} = 5\beta/\gamma$ shall not exceed β .

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

For the limit state of sidewall strength, the available strength for branches in tension shall be taken as the available strength for sidewall local yielding. For the limit state of sidewall strength, the available strength for branches in compression shall be taken as the lower of the strengths for sidewall local yielding and sidewall local crippling. For cross-connections with a branch angle less than 90° , an additional check for chord sidewall shear failure must be made in accordance with Section 10.7.5.

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

For the limit state of local yielding,

$$P_n \sin \theta = 2F_y t [5k + N] \quad (6.10.223)$$

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

Where,

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

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

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

$$P_n \sin \theta = 1.6t^2 \left[1 + \frac{3N}{(H-3t)} \right] (EF_y)^{0.5} Q_f \quad (6.10.224)$$

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

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

$$P_n \sin \theta = \left[\frac{48t^3}{(H-3t)} \right] (EF_y)^{0.5} Q_f \quad (6.10.225)$$

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

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

$$P_n = F_{yb} t_b [2H_b + 2b_{eoi} - 4t_b] \quad (6.10.226)$$

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

Where,

$$b_{eoi} = \left[\frac{10}{\left(\frac{B}{t}\right)} \right] [F_y t / (F_{yb} t_b)] B_b \leq B_b \quad (6.10.227)$$

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

10.11.2.3.3 Branches with axial loads in gapped K-connections

For gapped K-connections, the design strength of the branch, ϕP_n or the allowable strength of the branch, P_n/Ω , shall be the lowest value obtained according to the limit states of chord wall plastification, shear yielding (punching), shear yielding and local yielding due to uneven load distribution. In addition to the limits of applicability in Sec 10.11.2.3.1, the following limits shall apply:

$$(a) \quad B_b/B \geq 0.1 + \gamma/50$$

$$(b) \quad \beta_{eff} \geq 0.35$$

$$(c) \quad \zeta \leq 0.5(1 - \beta_{eff})$$

$$(d) \quad \text{Gap: } g \text{ greater than or equal to the sum of the branch wall thicknesses}$$

$$(e) \quad \text{The smaller } B_b > 0.63 \text{ times the larger } B_b$$

For the limit state of chord wall plastification,

$$P_n \sin \theta = F_y t^2 [9.8 \beta_{eff} \gamma^{0.5}] Q_f \quad (6.10.228)$$

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

For the limit state of shear yielding (punching),

$$P_n \sin \theta = 0.6 F_y t B [2\eta + \beta + \beta_{eop}] \quad (6.10.229)$$

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

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

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

For the limit state of shear yielding of the chord in the gap, available strength shall be checked in accordance with Sec 10.7. This limit state need only be checked if the chord is not square.

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

$$P_n = F_{yb} t_b [2H_b + B_b + b_{eoi} - 4t_b] \quad (6.10.230)$$

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

Where,

$$b_{eoi} = \left[\frac{10}{\left(\frac{B}{t}\right)} \right] [F_y t / (F_{yb} t_b)] B_b \leq B_b \quad (6.10.231)$$

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

10.11.2.3.4 Branches with axial loads in overlapped K-connections

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

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

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

$$P_n = F_{ybi} t_{bi} \left[\left(\frac{O_v}{50} \right) (2H_{bi} - 4t_{bi}) + b_{eoi} + b_{eov} \right] \quad (6.10.232)$$

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

$$P_n = F_{ybi} t_{bi} [2H_{bi} - 4t_{bi} + b_{eoi} + b_{eov}] \quad (6.10.233)$$

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

$$P_n = F_{ybi} t_{bi} [2H_{bi} - 4t_{bi} + B_{bi} + b_{eov}] \quad (6.10.234)$$

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

$$b_{eoi} = \left[\frac{10}{\left(\frac{B}{t} \right)} \right] [(F_y t) / (F_{ybi} t_{bi})] B_{bi} \leq B_{bi} \quad (6.10.235)$$

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

$$b_{eov} = \left[\frac{10}{\left(\frac{B_{bj}}{t_{bj}} \right)} \right] [(F_{ybj} t_{bj}) / (F_{ybi} t_{bi})] B_{bi} \leq B_{bi} \quad (6.10.236)$$

B_{bi} = overall branch width of the overlapping branch, mm.

B_{bj} = overall branch width of the overlapped branch, mm.

F_{ybi} = specified minimum yield stress of the overlapping branch material, MPa.

F_{ybj} = specified minimum yield stress of the overlapped branch material, MPa.

H_{bi} = overall depth of the overlapping branch, mm.

T_{bi} = thickness of the overlapping branch, mm.

t_{bj} = thickness of the overlapped branch, mm.

For the overlapped branch, P_n shall not exceed P_n of the overlapping branch, calculated using Eq. 6.10.232, Eq. 6.10.233 or Eq. 6.10.234, as applicable, multiplied by the factor $(A_{bj}F_{ybj}/A_{bi}F_{ybi})$.

Where,

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

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

10.11.2.3.5 Welds to branches

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

In T-, Y- and cross-connections,

For $\theta \geq 50^\circ$

$$L_e = \frac{2(H_b - 1.2t_b)}{\sin \theta} + (B_b - 1.2t_b) \quad (6.10.237)$$

For $\theta \geq 60^\circ$

$$L_e = \frac{2(H_b - 1.2t_b)}{\sin \theta} \quad (6.10.238)$$

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

In gapped K-connections, around each branch,

For $\theta \geq 50^\circ$

$$L_e = \frac{2(H_b - 1.2t_b)}{\sin \theta} + 2(B_b - 1.2t_b) \quad (6.10.239)$$

For $\theta \geq 60^\circ$

$$L_e = \frac{2(H_b - 1.2t_b)}{\sin \theta} + (B_b - 1.2t_b) \quad (6.10.240)$$

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

In lieu of the above criteria in Equations 6.10.237 to 6.10.240, other rational criteria are permitted.

10.11.3 HSS-To-HSS Moment Connections

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

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

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

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

10.11.3.1 Notation and definitions

The notation and their definitions related to this Section are given in Sec 10.1.2.

10.11.3.2 Criteria for round HSS

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

When the chord is in tension,

$$Q_f = 1$$

When the chord is in compression,

$$Q_f = 1.0 - 0.3U(1 + U) \quad (6.10.241)$$

Where, U is the utilization ratio given by

$$U = \left| \frac{P_r}{A_g F_c} + \frac{M_r}{S F_c} \right| \quad (6.10.242)$$

And,

P_r = required axial strength in chord, N.

M_r = required flexural strength in chord, N-mm.

A_g = chord gross area, mm²

F_c = available stress, MPa.

S = chord elastic section modulus, mm³

For design according to Section 10.2.3.3 (LRFD):

$P_r = P_u$ = required axial strength in chord, using LRFD load combinations, N

$M_r = M_u$ = required flexural strength in chord, using LRFD load combinations, N-mm.

$F_c = F_y$, MPa.

For design according to Section 10.2.3.4 (ASD):

$P_r = P_a$ = required axial strength in chord, using *ASD load combinations*, N

$M_r = M_a$ = required flexural strength in chord, using ASD load combinations, N-mm.

$F_c = 0.6F_y$, MPa.

10.11.3.2.1 Limits of applicability

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

- (a) Branch angle: $\theta \geq 30^\circ$
- (b) Chord wall slenderness: ratio of diameter to wall thickness less than or equal to 50 for T - and Y-connections; less than or equal to 40 for cross-connections
- (c) Tension branch wall slenderness: ratio of diameter to wall thickness less than or equal to 50
- (d) Compression branch wall slenderness: ratio of diameter to wall thickness less than or equal to $0.05 E/F_y$
- (e) Width ratio: $0.2 < D_b/D \leq 1.0$
- (f) Strength: $F_y \leq 360$ MPa. for chord and branches
- (g) Ductility: $F_y/F_u \leq 0.8$

10.11.3.2.2 Branches with In-plane bending moments in T-, Y- and cross-connections

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

For the limit state of chord plastification,

$$M_n \sin \theta = 5.39 F_y t^2 \gamma^{0.5} \beta D_b Q_f \quad (6.10.243)$$

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

For the limit state of shear yielding (punching),

$$M_n = 0.6 F_y t D_b^2 [(1 + 3 \sin \theta)/4 \sin^2 \theta] \quad (6.10.244)$$

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

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

10.11.3.2.3 Branches with out-of-plane bending moments in T-, Y- and cross-connections

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

For the limit state of chord plastification,

$$M_n \sin \theta = F_y t^2 D_b \left[\frac{3.0}{1 - 0.81\beta} \right] Q_f \quad (6.10.245)$$

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

For the limit state of shear yielding (punching),

$$M_n = 0.6 F_y t D_b^2 [(3 + \sin \theta)/4 \sin^2 \theta] Q_f \quad (6.10.246)$$

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

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

10.11.3.2.4 Branches with combined bending moment and axial force in T-, Y- and cross-connections

Connections subject to branch axial load, branch in-plane bending moment, and branch out-of-plane bending moment, or any combination of these load effects, should satisfy the following.

For design according to Section 10.2.3.3 (LRFD):

$$\left(\frac{P_r}{\phi P_n}\right) + \left(\frac{M_{r-ip}}{\phi M_{n-ip}}\right)^2 + (M_{r-op}/\phi M_{n-op}) \leq 1.0 \quad (6.10.247)$$

Where,

$P_r = P_u$ = required axial strength in branch, using LRFD load combinations, N

ϕP_n = design strength obtained from Sec 10.11.2.2.2

M_{r-ip} = required in-plane flexural strength in branch, using LRFD load combinations, N-mm.

ϕM_{n-ip} = design strength obtained from Sec 10.11.3.2.2

M_{r-op} = required out-of-plane flexural strength in branch, using LRFD load combinations, N-mm.

ϕM_{n-op} = design strength obtained from Sec 10.11.3.2.3

For design according to Section 10.2.3.4 (ASD):

$$\left(P_r/\left(\frac{P_n}{\Omega}\right)\right) + \left(\frac{M_{r-ip}}{\frac{M_{n-ip}}{\Omega}}\right)^2 + (M_{r-op}/(M_{n-op}/\Omega)) \leq 1.0 \quad (6.10.248)$$

Where,

$P_r = P_a$ = required axial strength in branch, using ASD load combinations, N

P_n/Ω = allowable strength obtained from Sec 10.11.2.2.2

M_{r-ip} = required in-plane flexural strength in branch, using ASD load combinations, N-mm.

M_{n-ip}/Ω = allowable strength obtained from Sec 10.11.3.2.2

M_{r-op} = required out-of-plane flexural strength in branch, using ASD load combinations, N-mm.

M_{n-op}/Ω = allowable strength obtained from Sec 10.11.3.2.3

10.11.3.3 Criteria for rectangular HSS

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

When the chord is in tension,

$$Q_f = 1$$

When the chord is in compression,

$$Q_f = \left(1.3 - \frac{0.4U}{\beta}\right) \leq 1 \quad (6.10.249)$$

Where, U is the utilization ratio given by

$$U = \left| \frac{P_r}{A_g F_c} + \frac{M_r}{S F_c} \right| \quad (6.10.250)$$

And,

P_r = required axial strength in chord, N.

M_r = required flexural strength in chord, N-mm.

A_g = chord gross area, mm²

F_c = available stress, MPa.

S = chord elastic section modulus, mm³.

For design according to Section 10.2.3.3 (LRFD):

$P_r = P_u$ = required axial strength in chord, using LRFD load combinations, N

$M_r = M_u$ = required flexural strength in chord, using LRFD load combinations, N-mm.

$F_c = F_y$, MPa.

For design according to Section 10.2.3.4 (ASD):

$P_r = P_a$ = required axial strength in chord, using ASD load combinations, N

$M_r = M_a$ = required flexural strength in chord, using ASD load combinations, N-mm.

$F_c = 0.6F_y$, MPa.

10.11.3.3.1 Limits of applicability

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

- (a) Branch angle is approximately 90°
- (b) Chord wall slenderness: ratio of overall wall width to thickness less than or equal to 35
- (c) Tension branch wall slenderness: ratio of overall wall width to thickness less than or equal to 35
- (d) Compression branch wall slenderness: ratio of overall wall width to thickness less than or equal to $1.25(E/F_{yb})^{0.5}$ and also less than 35
- (e) Width ratio: ratio of overall wall width of branch to overall wall width of chord greater than or equal to 0.25
- (f) Aspect ratio: $0.5 \leq \text{ratio of depth to width} \leq 2.0$
- (g) Strength: $F_y \leq 360 \text{ MPa}$. for chord and branches
- (h) Ductility: $F_y/F_u \leq 0.8$
- (i) Other limits apply for specific criteria

10.11.3.3.2 Branches with In-plane bending moments in T- and cross-connections

The design strength ϕM_n and the allowable strength, M_n/Ω , shall be the lowest value obtained according to the limit states of chord wall plastification, sidewall local yielding and local yielding due to uneven load distribution.

For the limit state of chord wall plastification,

$$M_n = F_y t^2 H_b \left[\left(\frac{1}{2\eta} \right) + \frac{2}{(1-\beta)^{0.5}} + \frac{\eta}{1-\beta} \right] Q_f \quad (6.10.251)$$

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

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

For the limit state of sidewall local yielding,

$$M_n = 0.5F_y^* t (H_b + 5t)^2 \quad (6.10.252)$$

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

Where,

$$F_y^* = F_y \text{ for T-connections}$$

$$F_y^* = 0.8 F_y \text{ for cross-connections}$$

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

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

$$M_n = F_{yb} \left[Z_b - \left(1 - \frac{b_{eoi}}{B_b} \right) B_b H_b t_b \right] \quad (6.10.253)$$

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

Where,

$$b_{eoi} = \left[\frac{10}{\left(\frac{B}{t} \right)} \right] [F_y t / (F_{yb} t_b)] B_b \leq B_b \quad (6.10.254)$$

Z_b = branch plastic section modulus about the axis of bending, mm³.

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

10.11.3.3.3 Branches with out-of-plane bending moments in T- and cross-connections

The design strength, ϕM_n and the allowable strength, M_n/Ω , shall be the lowest value obtained according to the limit states of chord wall plastification, sidewall local yielding, local yielding due to uneven load distribution and chord distortional failure.

For the limit state of chord wall plastification,

$$M_n = F_y t^2 \left[\frac{0.5H_b(1+\beta)}{(1-\beta)} + \left[\frac{2BB_b(1+\beta)}{1-\beta} \right]^{0.5} \right] Q_f \quad (6.10.255)$$

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

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

For the limit state of sidewall local yielding,

$$M_n = F_y^* t (B - t) (H_b + 5t) \quad (6.10.256)$$

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

Where,

$$F_y^* = F_y \text{ for T-connections}$$

$$F_y^* = 0.8 F_y \text{ for cross-connections}$$

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

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

$$M_n = F_{yb} \left[Z_b - 0.5 \left(1 - \frac{b_{eoi}}{B_b} \right)^2 B_b^2 t_b \right] \quad (6.10.257)$$

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

Where,

$$b_{eoi} = \left[\frac{10}{\left(\frac{B}{t} \right)} \right] [F_y t / (F_{yb} t_b)] B_b \leq B_b \quad (6.10.258)$$

Z_b = branch plastic section modulus about the axis of bending, mm³.

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

For the limit state of chord distortional failure,

$$M_n = 2F_y t [H_b t + [BHt(B + H)]^{0.5}] \quad (6.10.259)$$

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

This limit state need not be checked for cross-connections or for T-connections if chord distortional failure is prevented by other means.

10.11.3.3.4 Branches with combined bending moment and axial force in T- and cross-connections

Connections subject to branch axial load, branch in-plane bending moment and branch out-of-plane bending moment, or any combination of these load effects, should satisfy,

For design according to Section 10.2.3.3 (LRFD):

$$\left(\frac{P_r}{\phi P_n}\right) + \left(\frac{M_{r-ip}}{\phi M_{n-ip}}\right) + \left(\frac{M_{r-op}}{\phi M_{n-op}}\right) \leq 1.0 \quad (6.10.260)$$

Where,

$P_r = P_u$ = required axial strength in branch, using LRFD load combinations, N

ϕP_n = design strength obtained from Sec 10.11.2.3.2

M_{r-ip} = required in-plane flexural strength in branch, using LRFD load combinations, N-mm.

ϕM_{n-ip} = design strength obtained from Sec 10.11.3.3.2

M_{r-op} = required out-of-plane flexural strength in branch, using LRFD load combinations, N-mm.

ϕM_{n-op} = design strength obtained from Sec 10.11.3.3.3

For design according to Section 10.2.3.4 (ASD):

$$\left(\frac{P_r}{\frac{P_n}{\Omega}}\right) + \left(\frac{M_{r-ip}}{\frac{M_{n-ip}}{\Omega}}\right) + \left(\frac{M_{r-op}}{\frac{M_{n-op}}{\Omega}}\right) \leq 1.0 \quad (6.10.261)$$

Where,

$P_r = P_a$ = required axial strength in branch, using ASD load combinations, N

P_n/Ω = allowable strength obtained from Sec 10.11.2.3.2

M_{r-ip} = required in-plane flexural strength in branch, using ASD load combinations, N-mm.

M_{n-ip}/Ω = allowable strength obtained from Sec 10.11.3.3.2

M_{r-op} = required out-of-plane flexural strength in branch, using ASD load combinations, N-mm.

M_{n-op}/Ω = allowable strength obtained from Sec 10.11.3.3.3

10.12 Design for Serviceability

This Chapter addresses serviceability performance design requirements.

10.12.1 General Provisions

Serviceability is a state in which the function of a building, its appearance, maintainability, durability, and comfort of its occupants are preserved under normal usage. Limiting values of structural behavior for serviceability (for example, maximum deflections, accelerations) shall be chosen with due regard to the intended function of the structure. Serviceability shall be evaluated using appropriate load combinations for the serviceability limit states identified.

10.12.2 Camber

Where camber is used to achieve proper position and location of the structure, the magnitude, direction and location of camber shall be specified in the structural drawings in accordance with the provisions of Chapter 1.

10.12.3 Deflections

Deflections in structural members and structural systems under appropriate service load combinations shall not impair the serviceability of the structure. Limiting values of deflections of various structural members shall be in accordance with those specified in Sec 1.4 Chapter 1.

10.12.4 Drift

Drift of a structure shall be evaluated under service loads to provide for serviceability of the structure, including the integrity of interior partitions and exterior cladding. Drift under strength load combinations shall not cause collision with adjacent structures or exceed the limiting values specified in Sec 1.5.6 Chapter 1.

10.12.5 Vibration

The effect of vibration on the comfort of the occupants and the function of the structure shall be considered. Sources of vibration to be considered include pedestrian loading, vibrating machinery and others identified for the structure. It must be shown by any rational method of analysis that the vibrations induced by any source including the above mentioned ones is within tolerable limit and shall not cause any adverse effect on the safety, stability and durability of the structure.

10.12.6 Wind-Induced Motion

The effect of wind-induced motion of buildings on the comfort of occupants shall be considered. For flexible building and structures as defined in Sec. 2.4.2 Chapter 2, it must be shown by a rational dynamic analysis that wind induced vibration does not cause any discomfort to occupants as well as the wind induced dynamic effect does not cause any adverse effect on the safety, stability and durability of the structure.

10.12.7 Expansion and Contraction

The effects of thermal expansion and contraction of a building shall be considered. Damage to building *cladding* can cause water penetration and may lead to corrosion.

10.12.8 Connection Slip

The effects of connection slip shall be included in the design where slip at bolted connections may cause deformations that impair the serviceability of the structure. Where appropriate, the connection shall be designed to preclude slip. For the design of slip-critical connections see Sections 10.10.3.8 and 10.10.3.9.

10.13 Fabrication, Erection and Quality Control

This Chapter addresses requirements for design and shop drawings, fabrication, shop painting, erection and quality control.

10.13.1 Design Drawings and Specifications

Structural Design Drawings and Specifications

Unless otherwise indicated in the contract documents, the structural design drawings shall be based upon consideration of the design loads and forces to be resisted by structural steel frame in the completed project.

The structural design drawings shall clearly show the work that is to be performed and shall give the following information with sufficient dimensions to accurately convey the quantity and nature of the structural steel to be fabricated:

- (a) The size, section, material grade and location of all members;
- (b) All geometry and working points necessary for layout;
- (c) Floor elevations;
- (d) Column centers and offsets;

- (e) The camber requirements for members;
- (f) Joining requirements between elements of built-up members; and,
- (g) The information that is required in Sections 10.13.1.1.1 to 10.13.1.1.6.

The structural steel specifications shall include any special requirements for the fabrication and erection of the structural steel.

The structural design drawings, specifications and addenda shall be numbered and dated for the purposes of identification.

10.13.1.1 Detailing of components

Permanent bracing, column stiffeners, column web doubler plates, bearing stiffeners in beams and girders, web reinforcement, openings for other trades and other special details, where required, shall be shown in sufficient detail in the structural design drawings so that the quantity, detailing and fabrication requirements for these items can be readily understood.

10.13.1.2 Designer's responsibility

The owner's designated representative for design shall indicate one of the following options for each connection:

- (1) The complete connection design shall be shown in the structural design drawings;
- (2) In the structural design drawings or specifications, the connection shall be designated to be selected or completed by an experienced steel detailer; or,
- (3) In the structural design drawings or specifications, the connection shall be designated to be designed by a licensed professional engineer working for the fabricator.

In all of the above options,

- (a) The requirements of Sec 10.13.1.1 shall apply; and,
- (b) The approvals process in Sec 10.13.2.4 shall be followed.

When option (2) above is specified:

The experienced steel detailer shall utilize tables or schematic information provided in the structural design drawings in the selection or completion of the connections. When such information is not provided, standard reference information as approved by the owner's designated representative for design, shall be used.

When option (2) or (3) above is specified:

The owner's designated representative for design shall provide the following information in the structural design drawings and specifications:

- (a) Any restrictions on the types of connections that are permitted;
- (b) Data concerning the loads, including shears, moments, axial forces and transfer forces, that are to be resisted by the individual members and their connections, sufficient to allow the selection, completion, or design of the connection details while preparing the shop and erection drawings;
- (c) Whether the data required in (b) is given at the service-load level or the factored-load level;
- (d) Whether LRFD or ASD is to be used in the selection, completion, or design of connection details; and,
- (e) What substantiating connection information, if any, is to be provided with the shop and erection drawings to the owner's designated representative for design.

When option (3) above is specified:

- (a) The fabricator shall submit in a timely manner representative samples of the required substantiating connection information to the owner's designated representatives for design and construction. The owner's designated representative for design shall confirm in writing in a timely manner that these representative samples are consistent with the requirements in the contract documents, or shall advise what modifications are required to bring the representative samples into compliance with the requirements in the contract documents. This initial submittal and review is in addition to the requirements in Sec 10.13.2.4.
- (b) The Engineer in responsible charge of the connection design shall review and confirm in writing as part of the substantiating connection information, that the shop and erection drawings properly incorporate the connection designs. However, this review by the Engineer in responsible charge of the connection design does not replace the approval process of the shop and erection drawings by the owner's designated representative for design in Sec 10.13.2.4.
- (c) The fabricator shall provide a means by which the substantiating connection information is referenced to the related connections on the shop and erection drawings for the purpose of review.

10.13.1.2.1 Levelling plates

When leveling plates are to be furnished as part of the contract requirements, their locations and required thickness and sizes shall be specified in the contract documents.

10.13.1.2.2 Non-structural elements

When the structural steel frame, in the completely erected and fully connected state, requires interaction with non-structural steel elements for strength and/or stability, those non-structural steel elements shall be identified in the contract documents as required in Sec 10.13.5.10.

10.13.1.2.3 Camber

When camber is required, the magnitude, direction and location of camber shall be specified in the structural design drawings.

10.13.1.2.4 Painting information

Specific members or portions thereof that are to be left unpainted shall be identified in the contract documents. When shop painting is required, the painting requirements shall be specified in the contract documents, including the following information:

- (a) The identification of specific members or portions thereof to be painted;
- (b) The surface preparation that is required for these members;
- (c) The paint specifications and manufacturer's product identification that are required for these members; and,
- (d) The minimum dry-film shop-coat thickness that is required for these members.

10.13.1.3 Architectural, electrical and mechanical design drawings and specifications

All requirements for the quantities, sizes and locations of structural steel shall be shown or noted in the structural design drawings. The use of architectural, electrical and/or mechanical design drawings as a supplement to the structural design drawings is permitted for the purposes of defining detail configurations and construction information.

10.13.1.4 Discrepancies

When discrepancies exist between the design drawings and specifications, the design drawings shall govern. When discrepancies exist between scale dimensions in the design drawings and the figures written in them, the figures shall govern. When discrepancies exist between the structural design drawings and the architectural, electrical or mechanical design drawings or design drawings for other trades, the structural design drawings shall govern.

When a discrepancy is discovered in the contract documents in the course of the fabricator's work, the fabricator shall promptly notify the owner's designated representative for construction so that the discrepancy can be resolved by the owner's designated representative for design. Such resolution shall be timely so as not to delay the fabricator's work. See Sections 10.13.1.5 and 10.13.7.3.

10.13.1.5 Legibility of design drawings

Design drawings shall be clearly legible and drawn to an identified scale that is appropriate to clearly convey the information.

10.13.1.6 Revisions to the design drawings and specifications

Revisions to the design drawings and specifications shall be made either by issuing new design drawings and specifications or by reissuing the existing design drawings and specifications. In either case, all revisions, including revisions that are communicated through responses to RFIs or the annotation of shop and/or erection drawings (see Sec 10.13.2.4.2), shall be clearly and individually indicated in the contract documents. The contract documents shall be dated and identified by revision number. Each design drawings shall be identified by the same drawing number throughout the duration of the project, regardless of the revision. See also Sec 10.13.7.3.

10.13.2 Shop and Erection Drawings

Shop drawings shall be prepared in advance of fabrication and give complete information necessary for the fabrication of the component parts of the structure, including the location, type and size of welds and bolts. Erection drawings shall be prepared in advance of erection and give information necessary for erection of the structure. Shop and erection drawings shall clearly distinguish between shop and field welds and bolts and shall clearly identify pretensioned and slip-critical high-strength bolted connections. Shop and erection drawings shall be made with due regard to speed and economy in fabrication and erection.

10.13.2.1 Owner responsibility

The owner shall furnish, in a timely manner and in accordance with the contract documents, complete structural design drawings and specifications that have been released for construction. Unless otherwise noted, design drawings that are provided as part of a contract bid package shall constitute authorization by the owner that the design drawings are released for construction.

Records of the meetings should be written and distributed to all parties. Subsequent meetings to discuss progress and issues that arise during construction also can be helpful, particularly when they are held on a regular schedule.

10.13.2.2 Fabricator responsibility

Except as provided in Sec 10.13.2.5, the fabricator shall produce shop and erection drawings for the fabrication and erection of the structural steel and is responsible for the following :

- (a) The transfer of information from the contract documents into accurate and complete shop and erection drawings; and,
- (b) The development of accurate, detailed dimensional information to provide for the fit-up of parts in the field.

Each shop and erection drawing shall be identified by the same drawing number throughout the duration of the project and shall be identified by revision number and date, with each specific revision clearly identified.

When the fabricator submits a request to change connection details that are described in the contract documents, the fabricator shall notify the owner's designated representatives for design and construction in writing in advance of the submission of the shop and erection drawings. The owner's designated representative for design shall review and approve or reject the request in a timely manner.

When requested to do so by the owner's designated representative for design, the fabricator shall provide to the owner's designated representatives for design and construction its schedule for the submittal of shop and erection drawings so as to facilitate the timely flow of information between all parties.

10.13.2.3 Use of CAD files and/or copies of design drawings

The fabricator shall neither use nor reproduce any part of the design drawings as part of the shop or erection drawings without the written permission of the owner's designated representative for design. When CAD files or copies of the design drawings are made available for the fabricator's use, the fabricator shall accept this information under the following conditions:

- (a) All information contained in the CAD files or copies of the design drawings shall be considered instruments of service of the owner's designated representative for design and shall not be used for other projects, additions to the project or the completion of the project by others. CAD files and copies of the design drawings shall remain the property of the owner's designated representative for design and in no case shall the transfer of these CAD files or copies of design drawings be considered a sale.

- (b) The CAD files or copies of the design drawings shall not be considered to be contract documents. In the event of a conflict between the design drawings and the CAD files or copies thereof, the design drawings shall govern;
- (c) The use of CAD files or copies of the design drawings shall not in any way obviate the fabricator's responsibility for proper checking and coordination of dimensions, details, member sizes and fit-up and quantities of materials as required to facilitate the preparation of shop and erection drawings that are complete and accurate as required in Sec 4.2; and,
- (d) The fabricator shall remove information that is not required for the fabrication or erection of the structural steel from the CAD files or copies of the design drawings.

10.13.2.4 Approval

Except as provided in Sec 10.13.2.5, the shop and erection drawings shall be submitted to the owner's designated representatives for design and construction for review and approval. The shop and erection drawings shall be returned to the fabricator within 14 calendar days.

Final substantiating connection information, if any, shall also be submitted with the shop and erection drawings. The owner's designated representative for design is the final authority in the event of a disagreement between parties regarding connection design.

Approved shop and erection drawings shall be individually annotated by the owner's designated representatives for design and construction as either approved or approved subject to corrections noted. When so required, the fabricator shall subsequently make the corrections noted and furnish corrected shop and erection drawings to the owner's designated representatives for design and construction.

10.13.2.4.1 Constituents of approval

Approval of the shop and erection drawings, approval subject to corrections noted and similar approvals shall constitute the following:

- (a) Confirmation that the fabricator has correctly interpreted the contract documents in the preparation of those submittals;
- (b) Confirmation that the owner's designated representative for design has reviewed and approved the connection details shown on the shop and erection drawings and submitted in accordance with Sections 10.13.1 and 10.13.2, if applicable; and,

- (c) Release by the owner's designated representatives for design and construction for the fabricator to begin fabrication using the approved submittals.

Such approval shall not relieve the fabricator of the responsibility for either the accuracy of the detailed dimensions in the shop and erection drawings or the general fit-up of parts that are to be assembled in the field.

The fabricator shall determine the fabrication schedule that is necessary to meet the requirements of the contract.

10.13.2.4.2 Authorization by owner

Unless otherwise noted, any additions, deletions or revisions that are indicated in responses to RFIs or on the approved shop and erection drawings shall constitute authorization by the owner that the additions, deletions or revisions are released for construction. The fabricator and the erector shall promptly notify the owner's designated representative for construction when any direction or notation in responses to RFIs or on the shop or erection drawings or other information will result in an additional cost and/or a delay. See Sections 10.13.1.5 and 10.13.7.3.

10.13.2.5 Shop and/or erection drawings not furnished by the fabricator

When the shop and erection drawings are not prepared by the fabricator, but are furnished by others, they shall be delivered to the fabricator in a timely manner. These shop and erection drawings shall be prepared, insofar as is practical, in accordance with the shop fabrication and detailing standards of the fabricator. The fabricator shall neither be responsible for the completeness or accuracy of shop and erection drawings so furnished, nor for the general fit-up of the members that are fabricated from them.

10.13.2.6 The RFI process

When requests for information (RFIs) are issued, the process shall include the maintenance of a written record of inquiries and responses related to interpretation and implementation of the contract documents, including the clarifications and/or revisions to the contract documents that result, if any. RFIs shall not be used for the incremental release for construction of design drawings. When RFIs involve discrepancies or revisions, see Sections 10.13.1.3, 10.13.1.5, and 10.13.2.4.2.

10.13.2.7 Erection drawings

Erection drawings shall be provided to the erector in a timely manner so as to allow the erector to properly plan and perform the work.

10.13.3 Materials**10.13.3.1 Mill materials**

Unless otherwise noted in the contract documents, the fabricator is permitted to order the materials that are necessary for fabrication when the fabricator receives contract documents that have been released for construction.

Unless otherwise specified by means of special testing requirements in the contract documents, mill testing shall be limited to those tests that are required for the material in the ASTM specifications indicated in the contract documents. Materials ordered to special material requirements shall be marked by the supplier as specified in ASTM A6/A6M Section 12 prior to delivery to the fabricator's shop or other point of use. Such material not so marked by the supplier, shall not be used until:

- (a) Its identification is established by means of testing in accordance with the applicable ASTM specifications; and,
- (b) A fabricator's identification mark, as described in Sec 10.13.2 has been applied. When mill material does not satisfy ASTM A6/A6M tolerances for camber, profile, flatness or sweep, the fabricator shall be permitted to perform corrective procedures, including the use of controlled heating and/or mechanical straightening, subject to the limitations in the AISC Specification.

10.13.3.2 Stock materials

If used for structural purposes, materials that are taken from stock by the fabricator shall be of a quality that is at least equal to that required in the ASTM specifications indicated in the contract documents.

Material test reports shall be accepted as sufficient record of the quality of materials taken from stock by the fabricator. The fabricator shall review and retain the material test reports that cover such stock materials. However, the fabricator need not maintain records that identify individual pieces of stock material against individual material test reports, provided the fabricator purchases stock materials that meet the requirements for material grade and quality in the applicable ASTM specifications.

Stock materials that are purchased under no particular specification, under a specification that is less rigorous than the applicable ASTM specifications or without material test reports or other recognized test reports shall not be used without the approval of the owner's designated representative for design.

10.13.4 Fabrication

10.13.4.1 Cambering, curving and straightening

Local application of heat or mechanical means is permitted to be used to introduce or correct camber, curvature and straightness. The temperature of heated areas, as measured by approved methods, shall not exceed 593⁰ C for A514/A514M and A852/A852M steel nor 1,200⁰ F (649⁰ C) for other steels.

10.13.4.2 Thermal cutting

Thermally cut edges shall meet the requirements of AWS D1.1, Sections 5.15.1.2, 5.15.4.3 and 5.15.4.4 with the exception that thermally cut free edges that will be subject to calculated static tensile stress shall be free of round-bottom gouges greater than 5 mm deep and sharp V-shaped notches. Gouges deeper than 5 mm and notches shall be removed by grinding or repaired by welding.

Reentrant corners, except reentrant corners of *beam copes* and weld access holes, shall meet the requirements of AWS D1.1, Section A5.16. If another specified contour is required it must be shown on the contract documents.

Beam copes and weld access holes shall meet the geometrical requirements of Sec 10.10.1.6. Beam copes and weld access holes in shapes that are to be galvanized shall be ground. For shapes with a flange thickness not exceeding 50 mm the roughness of thermally cut surfaces of copes shall be no greater than a surface roughness value of 50 μ m as defined in ASME B46.1 Surface Texture (Surface Roughness, Waviness, and Lay). For beam copes and weld access holes in which the curved part of the access hole is thermally cut in ASTM A6/A6M hot-rolled shapes with a flange thickness exceeding 50 mm and welded built-up shapes with material thickness greater than 50 mm, a preheat temperature of not less than 66⁰ C shall be applied prior to thermal cutting. The thermally cut surface of access holes in ASTM A6/A6M hot-rolled shapes with a flange thickness exceeding 50 mm and built-up shapes with a material thickness greater than 50 mm shall be ground and inspected for cracks using magnetic particle inspection in accordance with ASTM E709. Any crack is unacceptable regardless of size or location.

10.13.4.3 Planing of edges

Planing or finishing of sheared or thermally cut edges of plates or shapes is not required unless specifically called for in the contract documents or included in a stipulated edge preparation for welding.

10.13.4.4 Welded construction

The technique of welding, the workmanship, appearance and quality of welds, and the methods used in correcting nonconforming work shall be in accordance with AWS D1.1 except as modified in Section J2.

10.13.4.5 Bolted construction

Parts of bolted members shall be pinned or bolted and rigidly held together during assembly. Use of a drift pin in bolt holes during assembly shall not distort the metal or enlarge the holes. Poor matching of holes shall be cause for rejection.

Bolt holes shall comply with the provisions of the RCSC Specification for Structural Joints Using ASTM A325 or A490 Bolts, Sec 10.13.3.3 except that thermally cut holes shall be permitted with a surface roughness profile not exceeding 25 μm as defined in ASME B46.1. Gouges shall not exceed a depth of 2 mm.

Fully inserted finger shims, with a total thickness of not more than 6 mm within a joint are permitted in joints without changing the strength (based upon hole type) for the design of connections. The orientation of such shims is independent of the direction of application of the load.

The use of high-strength bolts shall conform to the requirements of the RCSC Specification for Structural Joints Using ASTM A325 or A490 Bolts, except as modified in Sec 10.10.3.

10.13.4.6 Compression joints

Compression joints that depend on contact bearing as part of the splice strength shall have the bearing surfaces of individual fabricated pieces prepared by milling, sawing, or other suitable means.

10.13.4.7 Dimensional tolerances

Dimensional tolerances shall be in accordance with ASTM A6/A6M.

10.13.4.8 Finish of column bases

Column bases and base plates shall be finished in accordance with the following requirements:

- (1) Steel bearing plates 50 mm or less in thickness are permitted without milling, provided a satisfactory contact bearing is obtained. Steel bearing plates over 50 mm but not over 100 mm in thickness are permitted to be straightened by pressing or, if presses are not available, by milling for bearing surfaces (except as noted in subparagraphs 2 and 3 of

this Section), to obtain a satisfactory contact bearing. Steel bearing plates over 100 mm in thickness shall be milled for bearing surfaces (except as noted in subparagraphs 2 and 3 of this Section).

- (2) Bottom surfaces of bearing plates and column bases that are grouted to ensure full bearing contact on foundations need not be milled.
- (3) Top surfaces of bearing plates need not be milled when complete-joint- penetration groove welds are provided between the column and the bearing plate.

10.13.4.9 Holes for anchor rods

Holes for anchor rods shall be permitted to be thermally *cut* in accordance with the provisions of Sec 10.13.2.2.

10.13.4.10 Drain holes

When water can collect inside HSS or box members, either during construction or during service, the member shall be sealed, provided with a drain hole at the base, or protected by other suitable means.

10.13.4.11 Requirements for galvanized members

Members and parts to be galvanized shall be designed, detailed and fabricated to provide for flow and drainage of pickling fluids and zinc and to prevent pressure build-up in enclosed parts.

10.13.5 Shop Painting

10.13.5.1 General requirements

Shop paint is not required unless specified by the contract documents.

10.13.5.2 Inaccessible surfaces

Except for contact surfaces, surfaces inaccessible after shop assembly shall be cleaned and painted prior to assembly, if required by the design documents.

10.13.5.3 Contact surfaces

Paint is permitted in bearing-type connections. For slip-critical connections, the faying surface requirements shall be in accordance with the RCSC Specification for Structural Joints Using ASTM A325 or A490 Bolts, Section 3.2.2(b).

10.13.5.4 Finished surfaces

Machine-finished surfaces shall be protected against corrosion by a rust inhibitive coating that can be removed prior to erection, or which has characteristics that make removal prior to erection unnecessary.

10.13.5.5 Surfaces adjacent to field welds

Unless otherwise specified in the design documents, surfaces within 50 mm of any field weld location shall be free of materials that would prevent proper welding or produce objectionable fumes during welding.

10.13.6 Erection

10.13.6.1 Alignment of column bases

Column bases shall be set level and to correct elevation with full bearing on concrete or masonry.

10.13.6.2 Bracing

The frame of steel skeleton buildings shall be carried up true and plumb. Temporary bracing shall be provided, 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.

10.13.6.3 Alignment of structural elements

No permanent bolting or welding shall be performed until the adjacent affected portions of the structure have been properly aligned.

10.13.6.4 Fit of column compression joints and base plates

Lack of contact bearing not exceeding a gap of 2 mm, regardless of the type of splice used (partial-joint-penetration groove welded or bolted), is permitted. If the gap exceeds 2 mm, but is less than 6 mm, and if an engineering investigation shows that sufficient contact area does not exist, the gap shall be packed out with nontapered steel shims. Shims need not be other than mild steel, regardless of the grade of the main material.

10.13.6.5 Field welding

Shop paint on surfaces adjacent to joints to be field welded shall be wire brushed if necessary to assure weld quality.

Field welding of attachments to installed embedments in contact with concrete shall be done in such a manner as to avoid excessive thermal expansion of the embedment which could result in spalling or cracking of the concrete or excessive stress in the embedment anchors.

10.13.6.6 Field painting

Responsibility for touch-up painting, cleaning and field painting shall be allocated in accordance with accepted local practices, and this allocation shall be set forth explicitly in the design documents.

10.13.6.7 Connections

As erection progresses, the structure shall be securely bolted or welded to support dead, wind and erection loads.

10.13.7 Quality Control

The fabricator shall provide quality control procedures to the extent that the fabricator deems necessary to assure that the work is performed in accordance with this Specification. In addition to the fabricator's quality control procedures, material and workmanship at all times may be subject to inspection by qualified inspectors representing the purchaser. If such inspection by representatives of the purchaser will be required, it shall be so stated in the design documents.

10.13.7.1 Cooperation

As far as possible, the inspection by representatives of the purchaser shall be made at the fabricator's plant. The fabricator shall cooperate with the inspector, permitting access for inspection to all places where work is being done. The purchaser's inspector shall schedule this work for minimum interruption to the work of the fabricator.

10.13.7.2 Rejections

Material or workmanship not in conformance with the provisions of this Specification may be rejected at any time during the progress of the work.

The fabricator shall receive copies of all reports furnished to the purchaser by the inspection agency.

10.13.7.3 Inspection of welding

The inspection of welding shall be performed in accordance with the provisions of AWS D1.1 except as modified in Sec 10.10.2.

When visual inspection is required to be performed by AWS certified welding inspectors, it shall be so specified in the design documents.

When nondestructive testing is required, the process, extent and standards of acceptance shall be clearly defined in the design documents.

10.13.7.4 Inspection of slip-critical high-strength bolted connections

The inspection of slip-critical high-strength bolted connections shall be in accordance with the provisions of the RCSC Specification for Structural Joints Using ASTM A325 or A490 Bolts.

10.13.7.5 Identification of steel

The fabricator shall be able to demonstrate by a written procedure and by actual practice a method of material identification, visible at least to the “fit-up” operation, for the main structural elements of each shipping piece.

10.14 Direct Analysis Method

This Section addresses the direct analysis method for structural systems comprised of moment frames, braced frames, shear walls, or combinations thereof.

10.14.1 General Requirements

Members shall satisfy the provisions of Sec 10.8.1 with the nominal column strengths, P_n , determined using $K = 1.0$. The required strengths for members, connections and other structural elements shall be determined using a second-order elastic analysis with the constraints presented in Sec 10.14.3. All component and connection deformations that contribute to the lateral displacement of the structure shall be considered in the analysis.

10.14.2 Notional Loads

Notional loads shall be applied to the lateral framing system to account for the effects of geometric imperfections, inelasticity, or both. Notional loads are lateral loads that are applied at each framing level and specified in terms of the gravity loads applied at that level. The gravity load used to determine the notional load shall be equal to or greater than the gravity load associated with the load combination being evaluated. Notional loads shall be applied in the direction that adds to the destabilizing effects under the specified load combination.

10.14.3 Design-Analysis Constraints

- (1) The second-order analysis shall consider both $P-\delta$ and $P-\Delta$ effects. It is permitted to perform the analysis using any general second-order analysis method, or by the amplified first-order analysis method of Sec 10.3.2, provided that the B_1 and B_2

factors are based on the reduced stiffnesses defined in Eq. 6.10.263 and 6.10.264. Analyses shall be conducted according to the design and loading requirements specified in either Sec 10.2.3.3 (LRFD) or Sec 10.2.3.4 (ASD). For ASD, the second-order analysis shall be carried out under 1.6 times the ASD load combinations and the results shall be divided by 1.6 to obtain the required strengths.

Methods of analysis that neglect the effects of $P - \delta$ on the lateral displacement of the structure are permitted where the axial loads in all members whose flexural stiffnesses are considered to contribute to the lateral stability of the structure satisfy the following limit:

$$\alpha P_r < 0.15 P_{eL} \quad (6.10.262)$$

Where,

P_r = required axial compressive strength under LRFD or ASD load combinations,
N

$P_{eL} = \pi^2 EI / L^2$ evaluated in the plane of bending

And, $\alpha = 1.0$ (LRFD) $\alpha = 1.6$ (ASD)

- (2) A notional load, $N_i = 0.002Y_i$, applied independently in two orthogonal directions, shall be applied as a lateral load in all load combinations. This load shall be in addition to other lateral loads, if any,

Where,

N_i = notional lateral load applied at level i , N

Y_i = gravity load from LRFD load combination or 1.6 times the ASD load combination applied at level i , N

The notional load coefficient of 0.002 is based on an assumed initial story out-of-plumbness ratio of 1/500. Where a smaller assumed out-of-plumbness is justified, the notional load coefficient may be adjusted proportionally.

For frames where the ratio of second-order drift to first-order drift is equal to or less than 1.5, it is permissible to apply the notional load, N_i , as a minimum lateral load for the gravity-only load combinations and not in combination with other lateral loads.

For all cases, it is permissible to use the assumed out-of-plumbness geometry in the analysis of the structure in lieu of applying a notional load or a minimum lateral load as defined above.

- (3) A reduced flexural stiffness, EI^* ,

$$EI^* = 0.8\tau_b EI \quad (6.10.263)$$

shall be used for all members whose flexural stiffness is considered to contribute to the lateral stability of the structure,

Where,

I = moment of inertia about the axis of bending, mm^4

$\tau_b = 1.0$ for $\alpha P_r/P_y \leq 0.5$

$= 4[\alpha P_r/P_y(1 - \alpha P_r/P_y)]$ for $\alpha P_r/P_y > 0.5$

P_r = required axial compressive strength under *LRFD* or *ASD load combinations*, N

$P_y = AF_y$, member yield strength, N

And $\alpha = 1.0$ (LRFD) $\alpha = 1.6$ (ASD)

In lieu of using $\tau_b < 1.0$, where, $\alpha P_r/P_y > 0.5$, $\tau_b = 1.0$ may be used for all members, provided that an additive notional load of $0.001 Y_i$ is added to the notional load required in (2).

- (4) A reduced axial stiffness, EA^* ,

$$EA^* = 0.8 EA \quad (6.10.264)$$

shall be used for members whose axial stiffness is considered to contribute to the lateral stability of the structure, where A is the cross-sectional member area.

10.15 Inelastic Analysis and Design

10.15.1 General Provisions

Inelastic analysis is permitted for design according to the provisions of Sec 10.2.3.3 (LRFD). Inelastic analysis is not permitted for design according to the provisions of Sec 10.2.3.4 (ASD) except as provided in Sec 10.15.3.

10.15.2 Materials

Members undergoing plastic hinging shall have a specified minimum yield stress not exceeding 450 MPa.

10.15.3 Moment Redistribution

Beams and girders composed of compact sections as defined in Sec 10.2.4 and satisfying the unbraced length requirements of Sec 10.15.7, including composite members, may be proportioned for nine-tenths of the negative moments at points of support, produced by the gravity loading computed by an elastic analysis, provided that the maximum positive moment is increased by one-tenth of the average negative moments. This reduction is not permitted for moments produced by loading on cantilevers and for design according to Sections 10.15.4 to 10.15.8 of this Section.

If the negative moment is resisted by a *column* rigidly framed to the beam or girder, the one-tenth reduction may be used in proportioning the column for combined axial *force* and flexure, provided that the axial force does not exceed $1.5\phi_c F_y A_g$ for LRFD or $0.15F_y A_g / \Omega_c$ for ASD,

Where,

A_g = gross area of member, mm²

F_y = specified minimum yield stress of the compression flange, MPa.

ϕ_c = resistance factor for compression = 0.90

Ω_c = safety factor for compression = 1.67

10.15.4 Local Buckling

Flanges and webs of members subject to plastic hinging in combined flexure and axial compression shall be compact with width-thickness ratios less than or equal to the limiting λ_p defined in Table 6.10.1 or as modified as follows:

- (a) For webs of doubly symmetric wide flange members and rectangular HSS in combined flexure and compression

- (i) For $P_u/(\phi_b P_y) \leq 0.125$

$$h/tw \leq 3.76 \sqrt{\frac{E}{F_y}} \left(1 - \frac{2.75 P_u}{\phi_b P_y} \right) \quad (6.10.265)$$

- (ii) For $P_u/\phi_b P_y > 0.125$

$$h/tw \leq 1.12 \sqrt{\frac{E}{F_y}} \left(2.33 - \frac{P_u}{\phi_b P_y} \right) \geq 1.49 \sqrt{\frac{E}{F_y}} \quad (6.10.266)$$

Where,

E = modulus of elasticity of steel 200 000 MPa.

F_y = specified minimum yield stress of the type of steel being used, MPa.

h = as defined in Sec 10.2.4.2, mm.

P_u = required axial strength in compression, N.

P_y = member yield strength, N.

t_w = web thickness, mm.

ϕ_b = resistance factor for flexure = 0.90

- (b) For flanges of rectangular box and hollow structural sections of uniform thickness subject to bending or compression, flange cover plates, and diaphragm plates between lines of fasteners or welds

$$b/t \leq 0.94 \sqrt{E/F_y} \quad (6.10.267)$$

Where,

b = as defined in Sec 10.2.4.2, mm.

t = as defined in Sec 10.2.4.2, mm.

- (c) For circular hollow sections in flexure

$$D/t \leq 0.045 E/F_y \quad (6.10.268)$$

Where,

D = outside diameter of round HSS member, mm.

10.15.5 Stability and Second-Order Effects

Continuous beams not subjected to axial loads and that do not contribute to lateral stability of framed structures may be designed based on a first-order inelastic analysis or a plastic mechanism analysis.

Braced frames and moment frames may be designed based on a first-order inelastic analysis or a plastic mechanism analysis provided that stability and second-order effects are taken into account.

Structures may be designed on the basis of a second-order inelastic analysis. For beam-columns, connections and connected members, the required strengths shall be determined from a second-order inelastic analysis, where equilibrium is satisfied on deformed geometry, taking into account the change in stiffness due to yielding

10.15.5.1 Braced frames

In braced frames designed on the basis of inelastic analysis, braces shall be designed to remain elastic under the design loads. The required axial strength for columns and compression braces shall not exceed $\phi_c 0.85 F_y A_g$,

Where, $\phi_c = 0.90$ (LRFD)

10.15.5.2 Moment frames

In moment frames designed on the basis of inelastic analysis, the required axial strength of columns shall not exceed $\phi_c 0.75 F_y A_g$

Where, $\phi_c = 0.90$ (LRFD)

10.15.6 Columns and Other Compression Members

In addition to the limits set in Sections 10.15.5.1 and 10.15.5.2, the required axial strength of columns designed on the basis of inelastic analysis shall not exceed the design strength, $\phi_c P_n$, determined according to the provisions of Sec 10.5.3.

Design by inelastic analysis is permitted if the column slenderness ratio, L/r , does not exceed $4.71\sqrt{E/F_y}$,

Where,

L = laterally unbraced length of a member, mm.

r = governing radius of gyration, mm.

10.15.7 Beams and Other Flexural Members

The required moment strength, M_u , of beams designed on the basis of inelastic analysis shall not exceed the design strength, ϕM_n , where

$$M_n = M_p = F_y Z < 1.6 F_y S \quad (6.10.269)$$

$$\phi_c = 0.90 \text{ (LRFD)}$$

- (a) For doubly symmetric and singly symmetric I-shaped members with the compression flange equal to or larger than the tension flange loaded in the plane of the web:

$$L_{pd} = \left[0.12 + 0.076 \left(\frac{M_1}{M_2} \right) \right] \left(\frac{E}{F_y} \right) r_y \quad (6.10.270)$$

Where,

M_1 = smaller moment at end of unbraced length of beam, N-mm

M_2 = larger moment at end of unbraced length of beam, N-mm

r_y = radius of gyration about minor axis, mm

$(M_1 \times M_2)$ is positive when moments cause reverse curvature and negative for single curvature.

- (b) For solid rectangular bars and symmetric box beams:

$$L_{pd} = \left[0.17 + 0.10 \left(\frac{M_1}{M_2} \right) \right] \left(\frac{E}{F_y} \right) r_y \geq 0.10 \left(\frac{E}{F_y} \right) r_y \quad (6.10.271)$$

There is no limit on L_b for members with circular or square cross sections or for any beam bent about its minor axis.

10.15.8 Members under Combined Forces

When inelastic analysis is used for symmetric members subject to bending and axial force, the provisions in Sec 10.8.1 apply.

Inelastic analysis is not permitted for members subject to torsion and combined torsion, flexure, shear and/or axial force.

10.15.9 Connections

Connections adjacent to plastic hinging regions of connected members shall be designed with sufficient strength and ductility to sustain the forces and deformations imposed under the required loads.

10.16 Design for Ponding

This Section provides methods for determining whether a roof system has adequate strength and stiffness to resist ponding.

10.16.1 Simplified Design for Ponding

The roof system shall be considered stable for ponding and no further investigation is needed if both of the following two conditions are met:

$$C_p + 0.9 C_s \leq 0.25 \quad (6.10.272)$$

$$I_d \geq 3940 S^4 \quad (6.10.273)$$

Where,

$$C_p = \frac{504 L_s L_p^4}{I_p}$$

$$C_s = \frac{504 S L_s^4}{I_s}$$

L_p = column spacing in direction of girder (length of primary members), m.

L_s = column spacing perpendicular to direction of girder (length of secondary members), m.

S = spacing of secondary members, m.

I_p = moment of inertia of primary members, mm⁴.

I_s = moment of inertia of secondary members, mm⁴.

I_d = moment of inertia of the steel deck supported on secondary members, mm⁴ per m.

For trusses and steel joists, the moment of inertia I_s shall be decreased 15 percent when used in the above equation. A steel deck shall be considered a secondary member when it is directly supported by the primary members.

10.16.2 Improved Design for Ponding

The provisions given below are permitted to be used when a more exact determination of framing stiffness is needed than that given in Sec 10.16.1.

For primary members, the stress index shall be

$$U_p = \left(\frac{0.8F_y - f_o}{f_o} \right)_p \quad (6.10.274)$$

For secondary members, the stress index shall be

$$U_s = \left(\frac{0.8F_y - f_o}{f_o} \right)_s \quad (6.10.275)$$

Where,

f_o = stress due to the load combination ($D + R$)

D = nominal dead load

R = nominal load due to rainwater or snow, exclusive of the ponding contribution, MPa.

For roof framing consisting of primary and secondary members, the combined stiffness shall be evaluated as follows: enter Figure 6.10.1 at the level of the computed stress index U_p determined for the primary beam; move horizontally to the computed C_s value of the secondary beams and then downward to the abscissa scale. The combined stiffness of the primary and secondary framing is sufficient to prevent ponding if the flexibility constant read from this latter scale is more than the value of C_p computed for the given primary member; if not, a stiffer primary or secondary beam, or combination of both, is required.

A similar procedure must be followed using Figure 6.10.2.

For roof framing consisting of a series of equally spaced wall-bearing beams, the stiffness shall be evaluated as follows. The beams are considered as secondary members supported on an infinitely stiff primary member. For this case, enter Figure 6.10.2 with the computed stress index U_s . The limiting value of C_s is determined by the intercept of a horizontal line representing the U_s value and the curve for $C_p = 0$.

For roof framing consisting of metal deck spanning between beams supported on columns, the stiffness shall be evaluated as follows. Employ Figure 6.10.1 or 6.10.2 using as C_s the flexibility constant for a 1 m width of the roof deck ($S = 1.0$).

10.17 Design for Fatigue

This Section applies to members and connections subject to high cyclic loading within the elastic range of stresses of frequency and magnitude sufficient to initiate cracking and progressive failure, which defines the limit state of fatigue.

10.17.1 General

The provisions of this Section apply to stresses calculated on the basis of service loads. The maximum permitted stress due to unfactored loads is $0.66F_y$.

Stress range is defined as the magnitude of the change in stress due to the application or removal of the service live load. In the case of a stress reversal, the stress range shall be computed as the numerical sum of maximum repeated tensile and compressive stresses or the numerical sum of maximum shearing stresses of opposite direction at the point of probable crack initiation.

In the case of complete-joint-penetration butt welds, the maximum design stress range calculated by Eq. 6.10.276 applies only to welds with internal soundness meeting the acceptance requirements of Sec 6.12.2 or Sec 6.13.2 of AWS D1.1.

No evaluation of fatigue resistance is required if the live load stress range is less than the threshold stress range, FTH. See Table 6.10.14.

No evaluation of fatigue resistance is required if the number of cycles of application of live load is less than 20,000.

The cyclic load resistance determined by the provisions of this Section 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 Section is applicable only to structures subject to temperatures not exceeding 150°C.

The engineer of record shall provide either complete details including weld sizes or shall specify the planned cycle life and the maximum range of moments, shears and reactions for the connections.

10.17.2 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.

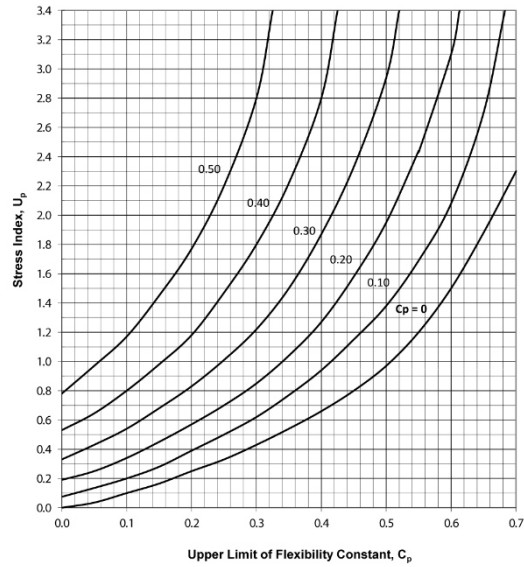


Figure 6.10.1 Limiting flexibility coefficient for the primary systems

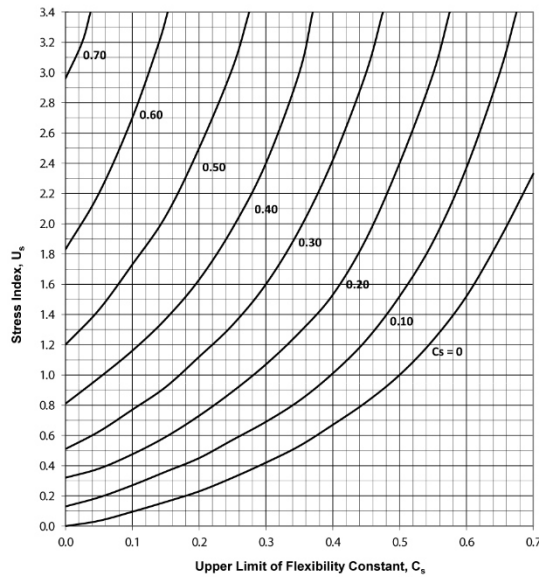


Figure 6.10.2 Limiting flexibility coefficient for the secondary systems.

10.17.3 Design Stress Range

The range of stress at service loads shall not exceed the design stress range computed as follows.

- (a) For stress categories A, B, B', C, D, E and E' (see Table 6.10.14) the design stress range, F_{SR} , shall be determined by Eq. 10.17.3.1 or 10.17.3.2.

$$F_{SR} = \left(\frac{C_f \times 329}{N} \right)^{0.333} \geq F_{TH} \quad (6.10.276)$$

Where,

F_{SR} = design stress range, MPa.

C_f = constant from Table 6.10.14 for the category

N = number of stress range fluctuations in design life

= number of stress range fluctuations per day $\times$ 365 $\times$ years of design life

F_{TH} = threshold fatigue stress (MPa) range, maximum stress range for indefinite design life from Table 6.10.14.

- (b) For stress category F, the design stress range, F_{SR} , shall be determined by Eq. 6.10.277.

$$F_{SR} = \left(\frac{C_f \times 11 \times 10^4}{N} \right)^{0.167} \geq F_{TH} \quad (6.10.277)$$

- (c) For tension-loaded plate elements connected at their end by cruciform, T, or corner details with complete-joint-penetration (CJP) groove welds or partial-joint-penetration (PJP) groove welds, fillet welds, or combinations of the preceding, transverse to the direction of stress, the design stress range on the cross section of the tension-loaded plate element at the toe of the weld shall be determined as follows:

- (i) Based upon crack initiation from the toe of the weld on the tension loaded plate element the design stress range, F_{SR} , shall be determined by Eq. 10.17.3.3 for stress category C which is equal to

$$F_{SR} = \left(\frac{14.4 \times 10^{11}}{N} \right)^{0.333} \geq 68.9 \quad (6.10.278)$$

- (ii) Based upon crack initiation from the root of the weld the design stress range, FSR, on the tension loaded plate element using transverse PJP groove welds, with or without reinforcing or contouring fillet welds, the design stress range on the cross section at the toe of the weld shall be determined by Eq. 10.17.3.4, stress category C' as follows:

$$F_{SR} = R_{PJP} \left(\frac{14.4 \times 10^{11}}{N} \right)^{0.333} \quad (6.10.279)$$

Where, R_{PJP} is the reduction factor for reinforced or nonreinforced transverse PJP groove welds determined as follows:

$$R_{PJP} = \left(\frac{1.12 - 1.01 \left(\frac{2a}{t_p} \right) + 1.24 \left(\frac{w}{t_p} \right)}{t_p^{0.167}} \right) \leq 1.0 \quad (6.10.280)$$

If $R_{PJP} = 1.0$, use stress category C.

$2a$ = the length of nonwelded root face in the direction of the thickness of the tension-loaded plate, mm.

W = the leg size of the reinforcing or contouring fillet, if any, in the direction of the thickness of the tension-loaded plate, mm.

t_p = thickness of tension loaded plate, mm.

- (iii) Based upon crack initiation from the roots of a pair of transverse fillet welds on opposite sides of the tension loaded plate element the design stress range, FSR, on the cross section at the toe of the welds shall be determined by Eq. 10.17.3.5, stress category C'' as follows:

$$F_{SR} = R_{FIL} \left(\frac{14.4 \times 10^{11}}{N} \right)^{0.333} \quad (6.10.281)$$

Where, R_{FIL} is the reduction factor for joints using a pair of transverse fillet welds only.

$$R_{FIL} = \left(\frac{0.10 + 1.24(w/t_p)}{t_p^{0.167}} \right) \leq 1.0 \quad (6.10.282)$$

If $R_{FIL} = 1.0$, use stress category C.

10.17.4 Bolts and Threaded Parts

The range of stress at service loads shall not exceed the stress range computed as follows.

- (a) For mechanically fastened connections loaded in shear, the maximum range of stress in the connected material at service loads shall not exceed the design stress range computed using Eq. 6.10.276 where C_f and FTH are taken from Section 2 of Table 6.10.14.
- (b) For high-strength bolts, common bolts, and threaded anchor rods with cut, ground or rolled threads, the maximum range of tensile stress on the net tensile area from applied axial load and moment plus load due to prying action shall not exceed the design stress range computed using Eq. 6.10.276. The factor C_f shall be taken as 3.9×10^8 (as for stress category E'). The threshold stress, FTH shall be taken as 48 MPa (as for stress category D). The net tensile area is given by Eq. 6.10.283.

$$A_t = \frac{\pi}{4} (d_b - 0.9382P)^2 \quad (6.10.283)$$

Where,

P = pitch, mm per thread

d_b = the nominal diameter (body or shank diameter), mm.

n = threads per mm.

For joints in which the material within the grip is not limited to steel or joints which are not tensioned to the requirements of Table 6.10.9, all axial load and moment applied to the joint plus effects of any prying action shall be assumed to be carried exclusively by the bolts or rods.

For joints in which the material within the grip is limited to steel and which are tensioned to the requirements of Table 6.10.9, an analysis of the relative stiffness of the connected parts and bolts shall be permitted to be used to determine the tensile stress range in the pretensioned bolts due to the total service live load and moment plus effects of any prying action. Alternatively, the stress range in the bolts shall be assumed to be equal to the stress on the net tensile area due to 20 percent of the absolute value of the service load axial load and moment from dead, live and other loads.

10.17.5 Special Fabrication and Erection Requirements

Longitudinal backing bars are permitted to remain in place, and if used, shall be continuous. If splicing is necessary for long joints, the bar shall be joined with complete penetration butt joints and the reinforcement ground prior to assembly in the joint.

Transverse joints subject to tension, backing bars, if used, shall be removed and joint back gouged and welded.

In transverse complete-joint-penetration T and corner joints, a reinforcing fillet weld, not less than 6 mm in size shall be added at re-entrant corners.

The surface roughness of flame cut edges subject to significant cyclic tensile stress ranges shall not exceed 25 μm , where ASME B46.1 is the reference standard.

Reentrant corners at cuts, *cope*s and weld access holes shall form a radius of not less than 10 mm by predrilling or subpunching and reaming a hole, or by thermal cutting to form the radius of the cut. If the radius portion is formed by thermal cutting, the cut surface shall be ground to a bright metal surface.

For transverse butt joints in regions of high tensile stress, run-off tabs shall be used to provide for cascading the weld termination outside the finished joint. End dams shall not be used. Run-off tabs shall be removed and the end of the weld finished flush with the edge of the member.

See Sec 10.10.2.2 for requirements for end returns on certain fillet welds subject to cyclic service loading.

Table 6.10.14a: Fatigue Design Parameters

Description	Stress Category	Constant C_f	Threshold F_{TH} (MPa)	Potential Crack Initiation Point
SECTION 1 – PLAIN MATERIAL AWAY FROM ANY WELDING				
1.1 Base metal, except non-coated weathering steel, with rolled or cleaned surface. Flame-cut edges with surface roughness value of 25 μm or less, but without reentrant corners.	A	250×10^8	165	Away from all welds or structural connections
1.2 Non-coated weathering steel base metal with rolled or cleaned surface. Flame-cut edges with surface roughness value of 25 μm or less, but without reentrant corners.	B	120×10^8	110	Away from all welds or structural connections

Description	Stress Category	Constant C_f	Threshold F_{TH} (MPa)	Potential Crack Initiation Point
1.3 Member with drilled or reamed holes. Member with reentrant corners at <i>copes</i> , cuts, block-outs or other geometrical discontinuities made to requirements of Sec 10.17.3.5, except weld access holes.	B	120×10^8	110	At any external edge or at hole perimeter
1.4 Rolled cross sections with weld access holes made to requirements of Sec 10.10.1.6 and Sec 10.17.3.5. Members with drilled or reamed holes containing bolts for attachment of light bracing where there is a small longitudinal component of brace force.	C	44×10^8	69	At reentrant corner of weld access hole or at any small hole (may contain bolt for minor connections)
SECTION 2– CONNECTED MATERIAL IN MECHANICALLY FASTENED JOINTS				
2.1 Gross area of base metal in lap joints connected by high-strength bolts in joints satisfying all requirements for slip- critical connections.	B	120×10^8	110	Through gross section near hole
2.2 Base metal at net section of high-strength bolted joints, designed on the basis of bearing resistance, but fabricated and installed to all requirements for slip-critical connections.	B	120×10^8	110	In net section originating at side of hole
2.3 Base metal at the net section of other mechanically fastened joints except eye bars and pin plates.	D	22×10^8	48	In net section originating at side of hole
2.4 Base metal at net section of eye bar head or pin plate.	E	11×10^8	31	In net section originating at side of hole

Description	Stress Category	Constant C_f	Threshold F_{TH} (MPa)	Potential Crack Initiation Point
SECTION 3 – WELDED JOINTS JOINING COMPONENTS OF BUILT-UP MEMBERS				
3.1 Base metal and weld metal in members without attachments built-up of plates or shapes connected by continuous longitudinal complete-joint-penetration groove welds, back gouged and welded from second side, or by continuous fillet welds.	B	120×10^8	110	From surface or internal discontinuities in weld away from end of weld
3.2 Base metal and weld metal in members without attachments built-up of plates or shapes, connected by continuous longitudinal complete-joint-penetration groove welds with backing bars not re-moved, or by continuous partial-joint-penetration groove welds.	B	61×10^8	83	From surface or internal discontinuities in weld, including weld attaching backing bars
3.3 Base metal and weld metal termination of longitudinal welds at weld access holes in connected built-up members.	D	22×10^8	48	From the weld termination into the web or flange
3.4 Base metal at ends of longitudinal intermittent fillet weld segments.	E	11×10^8	31	In connected material at start and stop locations of any weld deposit
3.5 Base metal at ends of partial length welded cover plates narrower than the flange having square or tapered ends, with or without welds across the ends of cover plates wider than the flange with welds across the ends.	E E'			In flange at toe of end weld or in flange at termination of longitudinal weld or in edge of flange with wide cover plates
Flange thickness ≤ 20 mm		11×10^8	31	
Flange thickness > 20 mm		3.9×10^8	18	

Description	Stress Category	Constant C_f	Threshold F_{TH} (MPa)	Potential Crack Initiation Point
3.6 Base metal at ends of partial length welded cover plates wider than the flange without welds across the ends.	E'	3.9×10^8	18	In edge of flange at end of cover plate weld
SECTION 4 – LONGITUDINAL FILLET WELDED END CONNECTIONS				
4.1 Base metal at junction of axially loaded members with longitudinally welded end connections. Welds shall be on each side of the axis of the member to balance weld stresses.				Initiating from end of any weld termination extending into the base metal.
$t \leq 20$ mm	E	11×10^8	31	
$t > 20$ mm	E'	3.9×10^8	18	
SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS				
5.1 Base metal and weld metal in or adjacent to complete-joint-penetration groove welded splices in rolled or welded cross sections with welds ground essentially parallel to the direction of stress.	B	120×10^8	110	From internal discontinuities in filler metal or along the fusion boundary
5.2 Base metal and weld metal in or adjacent to complete-joint-penetration groove welded splices with welds ground essentially parallel to the direction of stress at transitions in thickness or width made on a slope no greater than 8 to 20%.				From internal discontinuities in filler metal or along fusion boundary or at start of transition when $F_y \geq 620$ MPa
$F_y < 620$ MPa	B	120×10^8	110	
$F_y \geq 620$ MPa	B'	61×10^8	83	

Description	Stress Category	Constant C_f	Threshold F_{TH} (MPa)	Potential Crack Initiation Point
5.3 Base metal with F_y equal to or greater than 620 MPa and weld metal in or adjacent to complete-joint-penetration groove welded splices with welds ground essentially parallel to the direction of stress at transitions in width made on a radius of not less than 600 mm with the point of tangency at the end of the groove weld.	B	120×10^8	110	From internal discontinuities in filler metal or discontinuities along the fusion boundary
5.4 Base metal and weld metal in or adjacent to the toe of complete-joint-penetration T or corner joints or splices, with or without transitions in thickness having slopes no greater than 8 to 20%, when weld reinforcement is not removed.	C	44×10^8	69	From surface discontinuity at toe of weld extending into base metal or along fusion boundary.
5.5 Base metal and weld metal at transverse end connections of tension-loaded plate elements using partial-joint-penetration butt or T or corner joints, with reinforcing or contouring fillets, FSR shall be the smaller of the toe crack or root crack stress range.				Initiating from geometrical discontinuity at toe of weld extending into base metal or, initiating at weld root subject to tension extending up and then out through weld
Crack initiating from weld toe:	C	44×10^8	69	
Crack initiating from weld root:	C'	Eq. 6.10.279	None provided	

Description	Stress Category	Constant C_f	Threshold F_{TH} (MPa)	Potential Crack Initiation Point
5.6 Base metal and filler metal at transverse end connections of tension-loaded plate elements using a pair of fillet welds on opposite sides of the plate. FSR shall be the smaller of the toe crack or root crack stress range.				Initiating from geometrical discontinuity at toe of weld extending into base metal or, initiating at <i>weld root</i> subject to tension extending up and then out through weld
Crack initiating from weld toe:	C	44×10^8	69	
Crack initiating from weld root:	C'	Eq. 6.10.280	None provided	
5.7 Base metal of tension loaded plate elements and on girders and rolled beam webs or flanges at toe of transverse fillet welds adjacent to welded transverse stiffeners.	C	44×10^8	69	From geometrical discontinuity at <i>toe of fillet</i> extending into base metal
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS				
6.1 Base metal at details attached by complete joint penetration groove welds subject to longitudinal loading only when the detail embodies a transition radius R with the weld termination ground smooth.				Near point of tangency of radius at edge of member
$R \geq 600$ mm	B	120×10^8	110	
$600 \text{ mm} > R \geq 150$ mm	C	44×10^8	69	
$150 \text{ mm} > R \geq 50$ mm	D	22×10^8	48	
$50 \text{ mm} > R$	E	11×10^8	31	

Description	Stress Category	Constant C_f	Threshold F_{TH} (MPa)	Potential Crack Initiation Point
6.2 Base metal at details of equal thickness attached by complete-joint-penetration groove welds subject to trans-verse loading with or without longitudinal loading when the detail embodies a transition radius R with the weld termination ground smooth:				
When weld reinforcement is removed:				
$R \geq 600$ mm	B	120×10^8	110	Near points of tangency of radius or in the weld or at fusion boundary or member or attachment
$600 \text{ mm} > R \geq 150$ mm	C	44×10^8	69	
$150 \text{ mm} > R \geq 50$ mm	D	22×10^8	48	
$50 \text{ mm} > R$	E	11×10^8	31	
When weld reinforcement is not removed:				
$R \geq 600$ mm	C	44×10^8	69	At toe of the weld either along edge of member or the attachment
$600 \text{ mm} > R \geq 150$ mm	C	44×10^8	69	
$150 \text{ mm} > R \geq 50$ mm	D	22×10^8	48	
$50 \text{ mm} > R$	E	11×10^8	31	
6.3 Base metal at details of unequal thickness attached by complete-joint-penetration groove welds subject to trans-verse loading with or without longitudinal loading when the detail embodies a transition radius R with the weld termination ground smooth.				

Description	Stress Category	Constant C_f	Threshold F_{TH} (MPa)	Potential Crack Initiation Point
When weld reinforcement is removed: $R > 50$ mm	D	22×10^8	48	At toe of weld along edge of thinner material
$R \leq 50$ mm	E	11×10^8	31	In weld termination in small radius
When reinforcement is not removed: Any radius	E	11×10^8	31	At toe of weld along edge of thinner material
6.4 Base metal subject to longitudinal stress at transverse members, with or without transverse stress, attached by fillet or partial penetration groove welds parallel to direction of stress when the detail embodies a transition radius, R , with weld termination ground smooth:				In weld termination or from the toe of the weld extending into member
$R > 50$ mm	D	22×10^8	48	
$R \leq 50$ mm	E	11×10^8	31	
SECTION 7 – BASE METAL AT SHORT ATTACHMENTS¹				
7.1 Base metal subject to longitudinal loading at details attached by fillet welds parallel or transverse to the direction of stress where the detail embodies no transition radius and with detail length in direction of stress, a , and attachment height normal to the surface of the member, b :				In the member at the end of the weld

Description	Stress Category	Constant C_f	Threshold F_{TH} (MPa)	Potential Crack Initiation Point
$a < 50$ mm	C	44×10^8	69	
$50 \text{ mm} \leq a \leq 12b$ or 100 mm	D	22×10^8	48	
$a > 12b$ or 100 mm when b is ≤ 25 mm	E	11×10^8	31	
$a > 12b$ or 100 mm when b is > 25 mm	E'	3.9×10^8	18	
7.2 Base metal subject to longitudinal stress at details attached by fillet or partial-joint-penetration groove welds, with or without transverse load on detail, when the detail embodies a transition radius, R , with weld termination ground smooth:				In weld termination ex-tending into member
$R > 50$ mm	D	22×10^8	48	
$R \leq 50$ mm	E	11×10^8	31	
¹ “Attachment” as used herein, is defined as any steel detail welded to a member which, by its mere presence and independent of its loading, causes a discontinuity in the stress flow in the member and thus reduces the fatigue resistance.				
SECTION 8 - MISCELLANEOUS				
8.1 Base metal at stud-type shear connectors attached by fillet or electric stud welding.	C	44×10^8	69	At toe of weld in base metal
8.2 Shear on throat of continuous or intermittent longitudinal or transverse fillet welds.	F	150×10^{10} (Eq. 6.10.277)	55	In throat of weld
8.3 Base metal at plug or slot welds.	E	11×10^8	31	At end of weld in base metal
8.4 Shear on plug or slot welds.	F	150×10^{10} (Eq. 6.10.277)	55	At faying surface

Description	Stress Category	Constant C_f	Threshold F_{TH} (MPa)	Potential Crack Initiation Point
8.5 Not fully tightened high-strength bolts, common bolts, threaded anchor rods and hanger rods with cut, ground or rolled threads. Stress range on tensile stress area due to live load plus prying action when applicable.	E'	3.9×10^8	48	At the root of the threads extending into the tensile stress area

Table 6.10.14b: Fatigue Design Parameters (Illustrated Typical Examples)

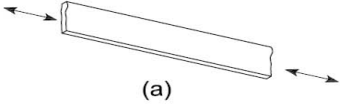
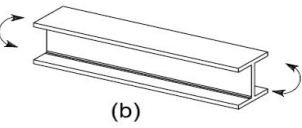
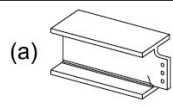
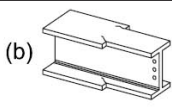
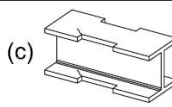
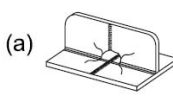
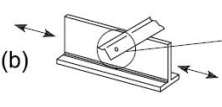
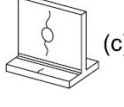
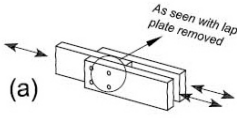
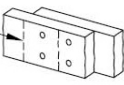
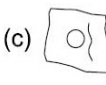
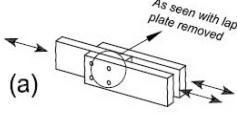
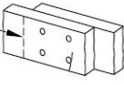
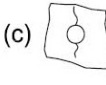
SECTION 1– PLAIN MATERIAL AWAY FROM ANY WELDING			
1.1 and 1.2			
1.3			
1.4			
SECTION 2– CONNECTED MATERIAL IN MECHANICALLY FASTENED JOINTS			
2.1			
2.2			

Table 6.10.14b: Fatigue Design Parameters (Illustrated Typical Examples)

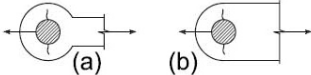
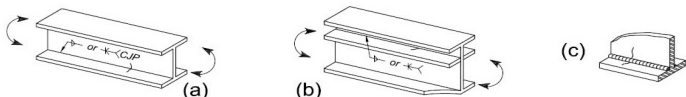
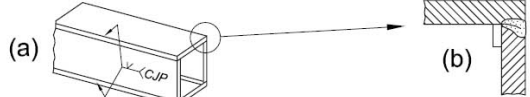
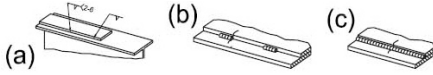
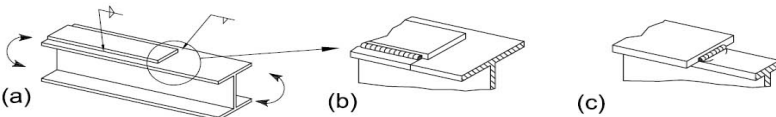
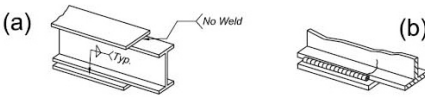
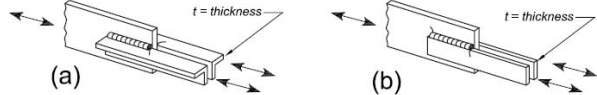
2.3	
2.4	
SECTION 3– WELDED JOINTS JOINING COMPONENTS OF BUILT-UP MEMBERS	
3.1	
3.2	
3.3	
3.4	
3.5	
3.6	
SECTION 4– LONGITUDINAL FILLET WELDED END CONNECTIONS	
4.1	

Table 6.10.14b: Fatigue Design Parameters (Illustrated Typical Examples)

SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS	
5.1	
5.2	$F_y \geq 90 \text{ ksi (620 MPa)}$ Cat. B'
5.3	$R \geq 2'-0" (600 \text{ mm})$ $F_y \geq 90 \text{ ksi (620 MPa)}$ Cat. B'
5.4	Cite for potential crack initiation due to bending tensile stress
5.5	Cite for potential crack initiation due to bending tensile stress $2a$
5.6	Potential crack due to bending tensile stress

Table 6.10.14b: Fatigue Design Parameters (Illustrated Typical Examples)

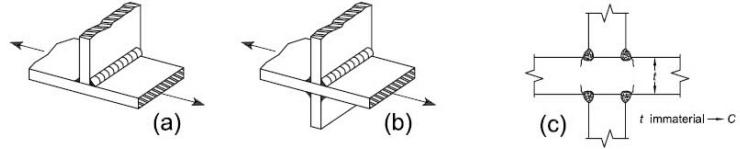
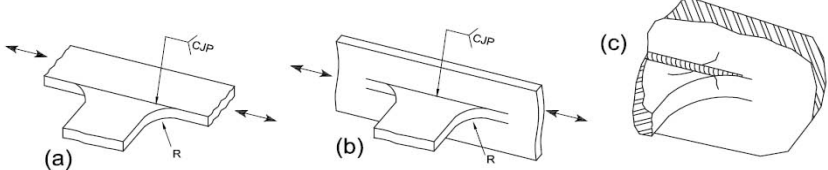
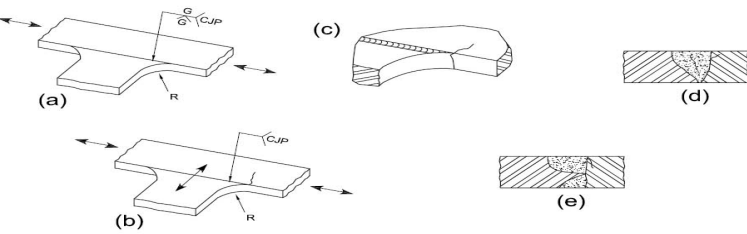
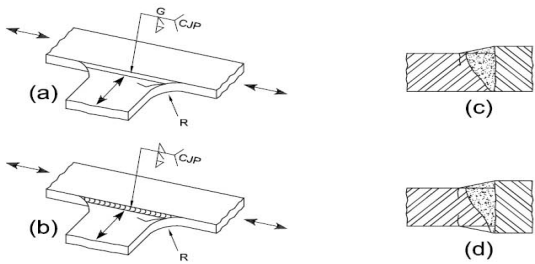
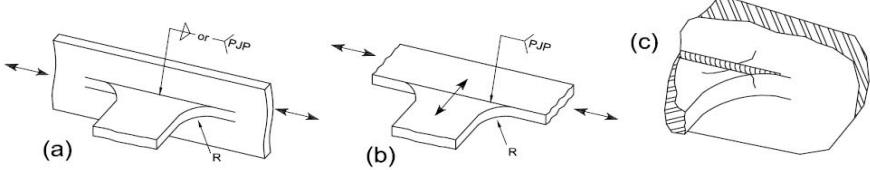
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SECTION 6- BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS	
6.1	
6.2	
6.3	
6.4	

Table 6.10.14b: Fatigue Design Parameters (Illustrated Typical Examples)

SECTION7– BASE METAL AT SHORT ATTACHMENTS1	
7.1	
7.2	
SECTION8- MISCELLANEOUS	
8.1	
8.2	
8.3	
8.4	
8.5	

10.18 Structural Design for Fire Conditions

This Section provides criteria for the design and evaluation of structural steel components, systems and frames for fire conditions. These criteria provide for the determination of the heat input, thermal expansion and degradation in mechanical properties of materials at elevated temperatures that cause progressive decrease in strength and stiffness of structural components and systems at elevated temperatures.

10.18.1 General Provisions

The methods contained in this Section provide regulatory evidence of compliance in accordance with the design applications outlined in this Section.

10.18.1.1 Performance objective

Structural components, members and building frame systems shall be designed so as to maintain their load-bearing function during the design-basis fire and to satisfy other performance requirements specified for the building occupancy. Deformation criteria shall be applied where the means of providing structural fire resistance, or the design criteria for fire barriers, requires consideration of the deformation of the load-carrying structure. Within the compartment of fire origin, forces and deformations from the design basis fire shall not cause a breach of horizontal or vertical compartmentation.

10.18.1.2 Design by engineering analysis

The analysis methods in Sec 10.18.2 are permitted to be used to document the anticipated performance of steel framing when subjected to design-basis fire scenarios. Methods in Sec 10.18.2 provide evidence of compliance with performance objectives established in Sec 10.18.1.1.

The analysis methods in Sec 10.18.2 are permitted to be used to demonstrate an equivalency for an alternative material or method, as permitted by the Code.

10.18.1.3 Design by qualification testing

The qualification testing methods in Sec 10.18.3 are permitted to be used to document the fire resistance of steel framing subject to the standardized fire testing protocols required by building Codes.

10.18.1.4 Load combinations and required strength

The required strength of the structure and its elements shall be determined from the following gravity load combination:

$$[0.9 \text{ or } 1.2]D + T + 0.5L \quad (6.10.284)$$

Where,

D = nominal dead load

L = nominal occupancy live load

T = nominal forces and deformations due to the design-basis fire defined in Section 4.2.1

A lateral notional load, $N_i = 0.002Y_i$, as defined in Sec 10.20, where N_i = notional lateral load applied at framing level i and Y_i = gravity load from combination 10.18.1.1 acting on framing level i , shall be applied in combination with the loads stipulated in Eq. 6.10.284. Unless otherwise stipulated by the Authority, D, L and S shall be the nominal loads specified in Chapter 2 of Part 6 of this Code.

10.18.2 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.

10.18.2.1 Design-basis fire

A design-basis fire shall be identified to describe the heating conditions for the structure. These heating conditions shall relate to the fuel commodities and compartment characteristics present in the assumed fire area. The fuel load density based on the occupancy of the space shall be considered when determining the total fuel load. Heating conditions shall be specified either in terms of a heat flux or temperature of the upper gas layer created by the fire. The variation of the heating conditions with time shall be determined for the duration of the fire.

When the analysis methods in Sec 10.18.2 are used to demonstrate an equivalency as an alternative material or method as permitted by Code, the design-basis fire shall be determined in accordance with ASTM E119.

10.18.2.1.1 Localized fire

Where the heat release rate from the fire is insufficient to cause flashover, a localized fire exposure shall be assumed. In such cases, the fuel composition, arrangement of the fuel array and floor area occupied by the fuel shall be used to determine the radiant heat flux from the flame and smoke plume to the structure.

10.18.2.1.2 Post-flashover compartment fires

Where the heat release rate from the fire is sufficient to cause flashover, a post-flashover compartment fire shall be assumed. The determination of the temperature versus time profile resulting from the fire shall include fuel load, ventilation characteristics to the space (natural and mechanical), compartment dimensions and thermal characteristics of the compartment boundary.

10.18.2.1.3 Exterior fires

The exposure of exterior structure to flames projecting from windows or other wall openings as a result of a post-flashover compartment fire shall be considered along with the radiation from the interior fire through the opening. The shape and length of the flame projection shall be used along with the distance between the flame and the exterior steelwork to determine the heat flux to the steel. The method identified in Sec 10.18.2.1.2 shall be used for describing the characteristics of the interior compartment fire.

10.18.2.1.4 Fire duration

The fire duration in a particular area shall be determined by considering the total combustible mass, in other words, fuel load available in the space. In the case of either a localized fire or a post-flashover compartment fire, the time duration shall be determined as the total combustible mass divided by the mass loss rate, except where determined from Sec 10.18.2.1.2.

10.18.2.1.5 Active fire protection systems

The effects of active fire protection systems shall be considered when describing the design-basis fire.

Where automatic smoke and heat vents are installed in nonsprinklered spaces, the resulting smoke temperature shall be determined from calculation.

10.18.2.2 Temperatures in structural systems under fire conditions

Temperatures within structural members, components and frames due to the heating conditions posed by the design-basis fire shall be determined by a heat transfer analysis.

10.18.2.3 Material strengths at elevated temperatures

Material properties at elevated temperatures shall be determined from test data. In the absence of such data, it is permitted to use the material properties stipulated in this Section. These relationships do not apply for steels with a yield strength in excess of 448 MPa or concretes with specified compression strength in excess of 55 MPa.

Table 6.10.15: Properties of Steel at Elevated Temperatures

Steel Temperature °C	$k_E = E_m/E$	$k_y = F_{ym}/F_y$	$k_u = F_{um}/F_y$
20	*	*	*
93	1.00	*	*
204	0.90	*	*

Steel Temperature °C	$k_E = E_m/E$	$k_y = F_{ym}/F_y$	$k_u = F_{um}/F_y$
316	0.78	*	*
399	0.70	1.00	1.00
427	0.67	0.94	0.94
538	0.49	0.66	0.66
649	0.22	0.35	0.35
760	0.11	0.16	0.16
871	0.07	0.07	0.07
982	0.05	0.04	0.04
1093	0.02	0.02	0.02
1204	0.00	0.00	0.00
*Use Ambient Properties			

10.18.2.3.1 Thermal elongation

Thermal expansion of structural and reinforcing steels: For calculations at temperatures above 65°C, the coefficient of thermal expansion shall be 1.4×10^{-5} per °C.

Thermal expansion of normal weight concrete: For calculations at temperatures above 65°C, the coefficient of thermal expansion shall be 1.8×10^{-5} per °C.

Thermal expansion of lightweight concrete: For calculations at temperatures above 65°C, the coefficient of thermal expansion shall be 7.9×10^{-6} per °C.

10.18.2.3.2 Mechanical properties at elevated temperatures

The deterioration in strength and stiffness of structural members, components, and systems shall be taken into account in the structural analysis of the frame.

The values F_{ym} , F_{um} , E_m , f'_{cm} , E_{cm} and ϵ_{cu} at elevated temperature to be used in structural analysis, expressed as the ratio with respect to the property at ambient, assumed to be 20°C, shall be defined as in Tables 6.10.15 and 6.10.16. It is permitted to interpolate between these values.

10.18.2.4 Structural design requirements

10.18.2.4.1 General structural integrity

The structural frame shall be capable of providing adequate strength and deformation capacity to withstand, as a system, the structural actions developed during the fire within the prescribed limits of deformation. The structural system shall be designed to sustain local damage with structural system as a whole remaining stable.

Continuous load paths shall be provided to transfer all forces from the exposed region to the final point of resistance. The foundation shall be designed to resist the forces and to accommodate the deformations developed during the design-basis fire.

Table 6.10.16: Properties of Concrete at Elevated Temperatures

Concrete Temperature °C	$k_c = f'_{cm}/f'_c$		E_{am}/E_c	ϵ_{cu} (%)
	NWC	LWC		LWC
20	1.00	1.00	1.00	0.25
93	0.95	1.00	0.93	0.34
204	0.90	1.00	0.75	0.46
288	0.86	1.00	0.61	0.58
316	0.83	0.98	0.57	0.62
427	0.71	0.85	0.38	0.80
538	0.54	0.71	0.20	1.06
649	0.38	0.58	0.092	1.32
760	0.21	0.45	0.073	1.43
871	0.10	0.31	0.055	1.49
982	0.05	0.18	0.036	1.50
1093	0.01	0.05	0.018	1.50
1204	0.00	0.00	0.00	-

10.18.2.4.2 Strength requirements and deformation limits

Conformance of the structural system to these requirements shall be demonstrated by constructing a mathematical model of the structure based on principles of structural mechanics and evaluating this model for the internal forces and deformations in the members of the structure developed by the temperatures from the design-basis fire.

Individual members shall be provided with adequate strength to resist the shears, axial forces and moments determined in accordance with these provisions.

Connections shall develop the strength of the connected members or the forces indicated above. Where the means of providing fire resistance requires the consideration of deformation criteria, the deformation of the structural system, or members thereof, under the design-basis fire shall not exceed the prescribed limits.

10.18.2.4.3 Methods of analysis

(a) Advanced methods of analysis

The methods of analysis in this Section are permitted for the design of all steel building structures for fire conditions. The design-basis fire exposure shall be that determined in Sec 10.18.2.1. The analysis shall include both a thermal response and the mechanical response to the design-basis fire.

The thermal response shall produce a temperature field in each structural element as a result of the design-basis fire and shall incorporate temperature- dependent thermal properties of the structural elements and fire-resistive materials as per Sec 10.18.2.2.

The mechanical response results in forces and deflections in the structural system subjected to the thermal response calculated from the design-basis fire. The mechanical response shall take into account explicitly the deterioration in strength and stiffness with increasing temperature, the effects of thermal expansions and large deformations. Boundary conditions and connection fixity must represent the proposed structural design. Material properties shall be defined as per Sec 10.18.2.3.

The resulting analysis shall consider all relevant limit states, such as excessive deflections, connection fractures, and overall or local buckling.

(b) Simple Methods of Analysis

The methods of analysis in this Section are applicable for the evaluation of the performance of individual members at elevated temperatures during exposure to fire. The support and restraint conditions (forces, moments and boundary conditions) applicable at normal temperatures may be assumed to remain unchanged throughout the fire exposure.

(1) Tension members

It is permitted to model the thermal response of a tension element using a one-dimensional heat transfer equation with heat input as directed by the design-basis fire defined in Sec 10.18.2.1.

The design strength of a tension member shall be determined using the provisions of Sec 10.4, with steel properties as stipulated in Sec 10.18.2.3 and assuming a uniform temperature over the cross section using the temperature equal to the maximum steel temperature.

(2) Compression members

It is permitted to model the thermal response of a compression element using a one-dimensional heat transfer equation with heat input as directed by the design-basis fire defined in Sec 10.18.2.1.

The design strength of a compression member shall be determined using the provisions of Sec 10.5 with steel properties as stipulated in Sec 10.18.2.3.

(3) Flexural members

It is permitted to model the thermal response of flexural elements using a one-dimensional heat transfer equation to calculate bottom flange temperature and to assume that this bottom flange temperature is constant over the depth of the member. The design strength of a flexural member shall be determined using the provisions of Sec 10.6 with steel properties as stipulated in Sec 10.18.2.3.

(4) Composite floor members

It is permitted to model the thermal response of flexural elements supporting a concrete slab using a one-dimensional heat transfer equation to calculate bottom flange temperature. That temperature shall be taken as constant between the bottom flange and mid-depth of the web and shall decrease linearly by no more than 25 percent from the mid-depth of the web to the top flange of the beam.

The design strength of a composite flexural member shall be determined using the provisions of Chapter 13 Part 6 of this Code, with reduced yield stresses in the steel consistent with the temperature variation described under thermal response.

10.18.2.4.4 Design strength

The design strength shall be determined as in Sec 10.2.3.3. The nominal strength, R_n shall be calculated using material properties, as stipulated in Sec 10.18.2.3, at the temperature developed by the design-basis fire.

10.18.3 Design by Qualification Testing**10.18.3.1 Design strength**

Structural members and components in steel buildings shall be qualified for the rating period in conformance with ASTM E119. It shall be permitted to demonstrate compliance with these requirements using the procedures specified for steel construction in Section 5 of ASCE/SFPE 29.

10.18.3.2 Restrained construction

For floor and roof assemblies and individual beams in buildings, a restrained condition exists when the surrounding or supporting structure is capable of resisting actions caused by thermal expansion throughout the range of anticipated elevated temperatures.

Steel beams, girders and frames supporting concrete slabs that are welded or bolted to integral framing members (in other words, columns, girders) shall be considered restrained construction.

10.18.3.3 Unrestrained construction

Steel beams, girders and frames that do not support a concrete slab shall be considered unrestrained unless the members are bolted or welded to surrounding construction that has been specifically designed and detailed to resist actions caused by thermal expansion.

A steel member bearing on a wall in a single span or at the end span of multiple spans shall be considered unrestrained unless the wall has been designed and detailed to resist effects of thermal expansion.

10.19 Stability Bracing for Columns and Beams

This Section addresses the minimum brace strength and stiffness necessary to provide member strengths based on the unbraced length between braces with an effective length factor, K , equal to 1.0.

10.19.1 General Provisions

Bracing is assumed to be perpendicular to the members to be braced; for inclined or diagonal bracing, the brace strength (force or moment) and stiffness (force per unit displacement or moment per unit rotation) shall be adjusted for the angle of inclination. The evaluation of the stiffness furnished by a brace shall include its member and geometric properties, as well as the effects of connections and anchoring details.

Two general types of bracing systems are considered, relative and nodal. A relative brace controls the movement of the brace point with respect to adjacent braced points. A nodal brace controls the movement at the braced point without direct interaction with adjacent braced points. The available strength and stiffness of the bracing shall equal or exceed the required limits unless analysis indicates that smaller values are justified by analysis.

A second-order analysis that includes an initial out-of-straightness of the member to obtain brace strength and stiffness is permitted in lieu of the requirements of this Section.

10.19.2 Columns

It is permitted to brace an individual column at end and intermediate points along its length by either relative or nodal bracing systems. It is assumed that nodal braces are equally spaced along the column.

10.19.2.1 Relative bracing

The required brace strength is

$$P_{br} = 0.004 P_r \quad (6.10.285)$$

The required brace stiffness is

$$\beta_{br} = \frac{1}{\phi} \left(\frac{2P_r}{L_b} \right) \text{ (LRFD)} \quad \beta_{br} = \Omega \left(\frac{2P_r}{L_b} \right) \text{ (ASD)} \quad (6.10.286)$$

Where,

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

For design according to Sec 10.2.3.3 (LRFD)

P_r = required axial compressive strength using LRFD load combinations, N

For design according to Sec 10.2.3.4 (ASD)

P_r = required axial compressive strength using ASD load combinations, N

10.19.2.2 Nodal bracing

The required brace strength is

$$P_{br} = 0.01 P_r \quad (6.10.287)$$

The required brace stiffness is

$$\beta_{br} = \frac{1}{\phi} \left(\frac{8P_r}{L_b} \right) \text{ (LRFD)} \quad \beta_{br} = \Omega \left(\frac{8P_r}{L_b} \right) \text{ (ASD)} \quad (6.10.288)$$

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

For design according to Sec 10.2.3.3 (LRFD)

P_r = required axial compressive strength using LRFD load combinations, N

For design according to Sec 10.2.3.4 (ASD)

P_r = required axial compressive strength using ASD load combinations, N

When, L_b is less than L_q , where L_q is the maximum unbraced length for the required column force with K equal to 1.0, then L_b in Eq. 6.10.288 is permitted to be taken equal to L_q .

10.19.3 Beams

At points of support for beams, girders and trusses, restraint against rotation about their longitudinal axis shall be provided. Beam bracing shall prevent the relative displacement of the top and bottom flanges, in other words, twist of the section. Lateral stability of beams shall be provided by lateral bracing, torsional bracing or a combination of the two. In members subjected to double curvature bending, the inflection point shall not be considered a brace point.

10.19.3.1 Lateral bracing

Bracing shall be attached near the compression flange, except for a cantilevered member, where an end brace shall be attached near the top (tension) flange. Lateral bracing shall be attached to both flanges at the brace point nearest the inflection point for beams subjected to double curvature bending along the length to be braced.

(a) Relative bracing

The required brace strength is

$$P_{br} = 0.008 M_r C_d / h_o \quad (6.10.289)$$

The required brace stiffness is

$$\beta_{br} = \frac{1}{\phi} \left(\frac{4M_r C_d}{L_b h_o} \right) \text{ (LRFD)} \quad \beta_{br} = \Omega \left(\frac{4M_r C_d}{L_b h_o} \right) \text{ (ASD)} \quad (6.10.290)$$

Where, $\phi = 0.75$ (LRFD) $\Omega = 2.00$ (ASD)

h_o = distance between flange centroids, mm.

C_d = 1.0 for bending in single curvature; 2.0 for double curvature; C_d = 2.0 only applies to the brace closest to the inflection point.

L_b = laterally unbraced length, mm.

For design according to Section 10.2.3.3 (LRFD):

M_r = required flexural strength using LRFD load combinations, N-mm

For design according to Section 10.2.3.4 (ASD):

M_r = required flexural strength using ASD load combinations, N-mm

(b) Nodal bracing

The required brace strength is

$$P_{br} = 0.02 M_r C_d / h_o \quad (6.10.291)$$

The required brace stiffness is

$$\beta_{br} = \frac{1}{\phi} \left(\frac{10 M_r C_d}{L_b h_o} \right) \text{ (LRFD)} \quad \beta_{br} = \Omega \left(\frac{10 M_r C_d}{L_b h_o} \right) \text{ (ASD)} \quad (6.10.292)$$

Where, $\phi = 0.75$ (LRFD) $\Omega = 2.00$ (ASD)

For design according to Section 10.2.3.3 (LRFD):

M_r = required flexural strength using LRFD load combinations, N-mm

For design according to Section 10.2.3.4 (ASD):

M_r = required flexural strength using ASD load combinations, N-mm

When L_b is less than L_q , the maximum unbraced length for M_r , then L_b in Eq. 6.10.292 shall be permitted to be taken equal to L_q .

10.19.3.2 Torsional bracing

It is permitted to provide either nodal or continuous torsional bracing along the beam length. It is permitted to attach the bracing at any cross-sectional location and it need not be attached near the compression flange. The connection between a torsional brace and the beam shall be able to support the required moment given below.

(a) Nodal bracing

The required bracing moment is

$$M_{br} = \frac{0.024 M_r L}{n C_b L_b} \quad (6.10.293)$$

The required cross-frame or diaphragm bracing stiffness is

$$\beta_{Tb} = \frac{\beta_T}{\left(1 - \frac{\beta_T}{\beta_{sec}}\right)} \quad (6.10.294)$$

Where,

$$\beta_T = \frac{1}{\phi} \left(\frac{2.4LM_r^2}{nEI_y C_b^2} \right) \text{ (LRFD)}$$

$$\beta_T = \Omega \left(\frac{2.4LM_r^2}{nEI_y C_b^2} \right) \text{ (ASD)} \quad (6.10.295)$$

$$\beta_{sec} = \frac{3.3 E}{h_o} \left(\frac{1.5h_o t_w^3}{12} + \frac{t_s b_s^3}{12} \right) \quad (6.10.296)$$

Where, $\phi = 0.75$ (LRFD) $\Omega = 3.00$ (ASD)

L = span length, mm

n = number of nodal braced points within the span

E = modulus of elasticity of steel 200000 MPa

I_y = out-of-plane moment of inertia, mm⁴

C_b = modification factor defined in Sec 10.6

t_w = beam web thickness, mm

t_s = web stiffener thickness, mm

b_s = stiffener width for one-sided stiffeners (use twice the individual stiffener width for pairs of stiffeners), mm.

β_T = brace stiffness excluding web distortion, N-mm/radian

β_{sec} = web distortional stiffness, including the effect of web transverse stiffeners, if any, N-mm/radian

For design according to Sec 10.2.3.3 (LRFD)

M_r = required flexural strength using LRFD load combinations, N-mm

For design according to Sec 10.2.3.4 (ASD)

M_r = required flexural strength using ASD load combinations, N-mm

If $\beta_{sec} < \beta_T$, Eq. 6.10.294 is negative, which indicates that torsional beam bracing will not be effective due to inadequate web distortional stiffness.

When required, the web stiffener shall extend the full depth of the braced member and shall be attached to the flange if the torsional brace is also attached to the flange. Alternatively, it shall be permissible to stop the stiffener short by a distance equal to $4t_w$ from any beam flange that is not directly attached to the torsional brace. When L_b is less than L_q then L_b in Eq. 6.10.293 shall be permitted to be taken equal to L_q .

(b) Continuous torsional bracing

For continuous bracing, use Equations 6.10.293, 6.10.294 and 6.10.296 with L/n taken as 1.0 and L_b taken as L_q ; bracing moment and stiffness are given per unit span length. The distortional stiffness for an unstiffened web is

$$\beta_{sec} = \frac{3.3 Et_w^3}{12h_o} \quad (6.10.297)$$

10.19.4 Slenderness Limitations

The slenderness ratio, L/r , of any stability bracing shall not exceed 180 unless a comprehensive analysis including second order effects justifies a higher value.

10.20 Seismic Provisions for Structural Steel Buildings

10.20.1 Scope

The Seismic Provisions for Structural Steel Buildings, hereinafter referred to as these Provisions as outline in this Sec 10.20, shall govern the design, fabrication and erection of structural steel members and connections in the seismic load resisting systems (SLRS) and splices in columns that are not part of the SLRS, in buildings and other structures, where other structures are defined as those structures designed, fabricated and erected in a manner similar to buildings, with building-like vertical and lateral load-resisting-elements.

These Provisions shall apply when the seismic response modification coefficient, R , (as specified in Chapter 2 of Part 6) is taken greater than 3, regardless of the seismic design category. When the seismic response modification coefficient, R , is taken as 3 or less, the structure is not required to satisfy the Provisions this Sec 10.20, unless specifically required by the applicable authority.

These Provisions shall be applied in conjunction with the specification set forth in Sections 10.1 to 10.19 whichever is applicable.

Loads, load combinations, system limitations and general design requirements shall be those in Chapter 2 Part 6 of this Code as well as those mentioned in Sec 10.2.

10.20.2 Referenced Specifications, Codes and Standards

The documents referenced in these Provisions shall include those listed in Sec 10.1.2 with the following additions and modifications:

American Institute of Steel Construction (AISC):

- Specification for Structural Steel Buildings, ANSI/AISC 360-05
- Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications, ANSI/AISC 358-05

American Society for Nondestructive Testing (ASNT):

- Recommended Practice for the Training and Testing of Nondestructive Testing Personnel, ASNT SNT TC-1a-2001
- Standard for the Qualification and Certification of Nondestructive Testing Personnel, ANSI/ASNT CP-189-2001

American Welding Society (AWS):

- Standard Methods for Determination of the Diffusible Hydrogen Content of Martensitic, Bainitic, and Ferritic Steel Weld Metal Produced by Arc Welding, AWS A4.3-93R
- Standard Methods for Mechanical Testing of Welds-U.S. Customary, ANSI/AWS B4.0-98
- Standard Methods for Mechanical Testing of Welds-Metric Only, ANSI/AWS B4.0M:2000
- Standard for the Qualification of Welding Inspectors, AWS B5.1:2003
- Oxygen Cutting Surface Roughness Gauge and Wall Chart for Criteria Describing Oxygen-Cut Surfaces, AWS C4.1

Federal Emergency Management Agency (FEMA)

- Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings, FEMA 350, July 2000

10.20.3 General Seismic Design Requirements

The required strength and other seismic provisions and the limitations on height and irregularity are specified in Chapter 2 Part 6 of this Code.

The design story drift shall be in accordance with the requirements set forth in Chapter 2 Part 6 of this Code.

10.20.4 Loads, Load Combinations, and Nominal Strengths**10.20.4.1 Loads and load combinations**

The loads and load combinations shall be as stipulated in Chapter 2 Part 6 of this Code. Where amplified seismic loads are required by these Provisions, the horizontal portion of the earthquake load E (as defined in Chapter 2 Part 6) shall be multiplied by the system over strength factor, Ω_o . The magnitude of over strength factor shall generally be obtained from Table 6.2.19 of Chapter 2 Part 6 of this Code. Reference may be made to Table 12.2-1 of ASCE 7-05 if data for a particular structure type is not found in Table 6.2.19. The value of Ω_o for a structural system shall be taken as 2.0 if it is not specified elsewhere.

10.20.4.2 Nominal strength

The nominal strength of systems, members and connections shall comply with the Specification, except as modified throughout these Provisions.

10.20.5 Structural Design Drawings and Specifications, Shop Drawings, and Erection Drawings**10.20.5.1 Structural design drawings and specifications**

Structural design drawings and specifications shall show the work to be performed, and include items required by the Specification and the following, as applicable:

- (1) Designation of the seismic load resisting system (SLRS)
- (2) Designation of the members and connections that are part of the SLRS
- (3) Configuration of the connections
- (4) Connection material specifications and sizes
- (5) Locations of demand critical welds
- (6) Lowest Anticipated Service Temperature (LAST) of the steel structure, if the structure is not enclosed and maintained at a temperature of 10° C or higher.
- (7) Locations and dimensions of protected zones
- (8) Locations where gusset plates are to be detailed to accommodate inelastic rotation
- (9) Welding requirements as specified in Appendix S, Sec S.2.1.

10.20.5.2 Shop drawings

Shop drawings shall include items required by the Specification and the following, as applicable:

- (1) Designation of the members and connections that are part of the SLRS
- (2) Connection material specifications
- (3) Locations of demand critical shop welds
- (4) Locations and dimensions of protected zones
- (5) Gusset plates drawn to scale when they are detailed to accommodate inelastic rotation
- (6) Welding requirements as specified in Appendix S, Sec S.2.2.

10.20.5.3 Erection drawings

Erection drawings shall include items required by the Specification and the following, as applicable:

- (1) Designation of the members and connections that are part of the SLRS
- (2) Field connection material specifications and sizes
- (3) Locations of demand critical field welds
- (4) Locations and dimensions of protected zones
- (5) Locations of pretensioned bolts
- (6) Field welding requirements as specified in Appendix S, Sec S.2.3

10.20.6 Materials

10.20.6.1 Material specifications

Structural steel used in the seismic load resisting system (SLRS) shall meet the requirements of Sec 10.1.3.1a, except as modified in present Sec 10.20. The specified minimum yield stress of steel to be used for members in which inelastic behavior is expected shall not exceed 345 MPa for systems defined in Sections 10.20.9, 10.20.10, 10.20.12, 10.20.13, 10.20.15, 10.20.16, and 10.20.17 nor 380 MPa for systems defined in Sections 10.20.11 and 10.20.14, unless the suitability of the material is determined by testing or other rational criteria. This limitation does not apply to columns for which only expected inelastic behavior is yielding at column base.

The structural steel used in the SLRS described in Sections 10.20.9 to 10.20.17 shall meet one of the following ASTM Specifications: A36/ A36M, A53/A53M, A500 (Grade B or C), A501, A529/A529M, A572/A572M [Grade 290, 345 or 380], A588/A588M, A913/A913M [Grade 345, 415 or 450], A992/A992M, or A1011 HSLAS Grade 380. The structural steel used for column base plates shall meet one of the preceding ASTM specifications or ASTM A283/A283M Grade D.

Other steels and non-steel materials in buckling-restrained braced frames are permitted to be used subject to the requirements of Sec 10.20.16 and Appendix R.

10.20.6.2 Material properties for determination of required strength of members and connections

The required strength of an element (a member or a connection) shall be determined from the expected yield stress, $R_y F_y$, of an adjoining member, where F_y is the specified minimum yield stress of the grade of steel to be used in the adjoining members and R_y is the ratio of the expected yield stress to the specified minimum yield stress, F_y , of that material.

The available strength of the element, ϕR_n for LRFD and R_n/Ω for ASD, shall be equal to or greater than the required strength, where R_n is the nominal strength of the connection. The expected tensile strength, $R_t F_u$, and the expected yield stress, $R_y F_y$, are permitted to be used in lieu of F_u and F_y , respectively, in determining the nominal strength, R_n , of rupture and yielding limit states within the same member for which the required strength is determined.

The values of R_y and R_t for various steels are given in Table 6.10.17. Other values of R_y and R_t shall be permitted if the values are determined by testing of specimens similar in size and source conducted in accordance with the requirements for the specified grade of steel.

10.20.6.3 Heavy section CVN requirements

For structural steel in the SLRS, in addition to the requirements of Sec 10.1.3.1c, hot rolled shapes with flange thickness 38 mm and thicker shall have a minimum Charpy V-Notch toughness of 27 J at 21°C, tested in the alternate core location as described in ASTM A6 Supplementary Requirement S30. Plates 50 mm thick and thicker shall have a minimum Charpy V-Notch toughness of 27 J at 21°C, measured at any location permitted by ASTM A673, where the plate is used in the following:

- (1) Members built-up from plate
- (2) Connection plates where inelastic strain under seismic loading is expected
- (3) As the steel core of buckling-restrained braces.

10.20.7 Connections, Joints and Fasteners**10.20.7.1 Scope**

Connections, joints and fasteners that are part of the Seismic Load Resisting System (SLRS) shall comply with Sec 10.10, and with the additional requirements of this Section.

The design of connections for a member that is a part of the SLRS shall be configured such that a ductile limit state in either the connection or the member controls the design.

10.20.7.2 Bolted joints

All bolts shall be pre-tensioned high strength bolts and shall meet the requirements for slip-critical faying surfaces in accordance with Sec 10.10.3.8 with a Class A surface. Bolts shall be installed in standard holes or in short-slotted holes perpendicular to the applied load. For brace diagonals, oversized holes shall be permitted when the connection is designed as a slip-critical joint, and the oversized hole is in one ply only. Alternative hole types are permitted if determined in a connection prequalification in accordance with Appendix N, or if determined in a program of qualification testing in accordance with Appendix Q or R. The available shear strength of bolted joints using standard holes shall be calculated as that for bearing-type joints in accordance with Sections 10.10.3.7 and 10.10.3.10, except that the nominal bearing strength at bolt holes shall not be taken greater than $2.4dtF_u$.

Exception: The faying surfaces for end plate moment connections are permitted to be coated with coatings not tested for slip resistance, or with coatings with a slip coefficient less than that of a Class A faying surface.

Bolts and welds shall not be designed to share force in a joint or the same force component in a connection.

Table 6.10.17: R_y and R_t Values for Different Member Types

Application	R_y	R_t
Hot-rolled structural shapes and bars:		
• ASTM A36/A36M	1.5	1.2
• ASTM A572/572M Grade 42 (290)	1.3	1.1
• ASTM A572/572M Grade 50 (345) or 55 (380), ASTM A913/A913M Grade 50 (345), 60 (415), or 65 (450), ASTM A588/A588M, ASTM A992/A992M, A1011 HSLAS Grade 55 (380)	1.1	1.1

Application	R_y	R_t
• ASTM A529 Grade 50 (345)	1.2	1.2
• ASTM A529 Grade 55 (380)	1.1	1.2
Hollow structural sections (HSS):		
• ASTM A500 (Grade B or C), ASTM A501	1.4	1.3
Pipe:		
• ASTM A53/A53M	1.6	1.2
Plates:		
• ASTM A36/A36M	1.3	1.2
• ASTM A572/A572M Grade 50 (345), ASTM A588/A588M	1.1	1.2

10.20.7.3 Welded Joints

Welding shall be performed in accordance with Appendix S. Welding shall be performed in accordance with a welding procedure specification (WPS) as required in AWS D1.1. The WPS variables shall be within the parameters established by the filler metal manufacturer.

10.20.7.3.1 General requirements

All welds used in members and connections in the SLRS shall be made with a filler metal that can produce welds that have a minimum Charpy V-Notch toughness of 27 J at minus 18°C, as determined by the appropriate AWS A5 classification test method or manufacturer certification. This requirement for notch toughness shall also apply in other cases as required in these Provisions.

10.20.7.3.2 Demand critical welds

Where welds are designated as demand critical, they shall be made with a filler metal capable of providing a minimum Charpy V-Notch (CVN) toughness of 27 J at 29°C as determined by the appropriate AWS classification test method or manufacturer certification, and 54 J at 21°C as determined by Appendix T or other approved method, when the steel frame is normally enclosed and maintained at a temperature of 10°C or higher. For structures with service temperatures lower than 10°C, the qualification temperature for Appendix T shall be 11°C above the lowest anticipated service temperature, or at a lower temperature.

SMAW electrodes classified in AWS A5.1 as E7018 or E7018-X, SMAW electrodes classified in AWS A5.5 as E7018-C3L or E8018-C3, and GMAW solid electrodes are exempted from production lot testing when the CVN toughness of the electrode equals or exceeds 27 J at a temperature not exceeding 29°C as determined by AWS classification test methods. The manufacturer's certificate of compliance shall be considered sufficient evidence of meeting this requirement.

10.20.7.3.3 Protected zone

Where a protected zone is designated by these Provisions, it shall comply with the following:

- (1) Within the protected zone, discontinuities created by fabrication or erection operations, such as tack welds, erection aids, air-arc gouging and thermal cutting shall be repaired as required by the engineer of record.
- (2) Welded shear studs and decking attachments that penetrate the beam flange shall not be placed on beam flanges within the protected zone. Decking arc spot welds as required to secure decking shall be permitted.
- (3) Welded, bolted, screwed or shot-in attachments for perimeter edge angles, exterior facades, partitions, duct work, piping or other construction shall not be placed within the protected zone.

Exception:

Welded shear studs and other connections shall be permitted when determined in accordance with a connection prequalification in accordance with Appendix N, or as determined in a program of qualification testing in accordance with Appendix Q.

Outside the protected zone, calculations based upon the expected moment shall be made to demonstrate the adequacy of the member net section when connectors that penetrate the member are used.

10.20.7.3.4 Continuity plates and stiffeners

Corners of continuity plates and stiffeners placed in the webs of rolled shapes shall be clipped as described below. Along the web, the clip shall be detailed so that the clip extends a distance of at least 38 mm beyond the published k detail dimension for the rolled shape. Along the flange, the clip shall be detailed so that the clip does not exceed a distance of 12 mm beyond the published k_1 detail dimension. The clip shall be detailed to facilitate suitable weld terminations for both the flange weld and the web weld. If a curved clip is used, it shall have a minimum radius of 12 mm.

At the end of the weld adjacent to the column web/flange juncture, weld tabs for continuity plates shall not be used, except when permitted by the engineer of record. Unless specified by the engineer of record that they be removed, weld tabs shall not be removed when used in this location.

10.20.8 Members**10.20.8.1 Scope**

Members in the seismic load resisting system (SLRS) shall comply with the specifications of Sections 10.1 to 10.11 and Sec 10.20.8. For columns that are not part of the SLRS, see Sec 10.20.8.4.2.

10.20.8.2 Classification of sections for local buckling**10.20.8.2.1 Compact**

When required by these Provisions, members of the SLRS shall have flanges continuously connected to the web or webs and the width-thickness ratios of its compression elements shall not exceed the limiting width-thickness ratios, λ_p , from Specification Table 6.10.1.

10.20.8.2.2 Seismically compact

When required by these Provisions, members of the SLRS must have flanges continuously connected to the web or webs and the width-thickness ratios of its compression elements shall not exceed the limiting width-thickness ratios, λ_{ps} , from Provisions Table 6.10.1.

10.20.8.3 Column strength

When $\frac{P_u}{\phi_c P_n}$ (LRFD) > 0.4 or $\frac{\Omega_c P_a}{P_n}$ (ASD) > 0.4 , as appropriate, without consideration of amplified seismic load,

Where,

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

P_a = required axial strength of a column using ASD load combinations, N

P_n = nominal axial strength of a column, N

P_u = required axial strength of a column using LRFD load combinations, N

The following requirements shall be met:

- (1) The required axial compressive and tensile strength, considered in the absence of any applied moment, shall be determined using load combinations stipulated by the Code including amplified seismic load.
- (2) The required axial compressive and tensile strength shall not exceed either of the following:
 - (a) The maximum load transferred to the column considering $1.1R_y$ (LRFD) or $\left(\frac{1.1}{1.5}\right)R_y$ (ASD), as appropriate, times the nominal strengths of the connecting beam or brace elements of the building.
 - (b) The limit as determined from the resistance of the foundation to over-turning uplift.

10.20.8.4 Column splices

10.20.8.4.1 General

The required strength of column splices in the seismic load resisting system (SLRS) shall equal the required strength of the columns, including that determined from Sections 10.20.8.3, 10.20.9.9, 10.20.10.9, 10.20.11.9, 10.20.13.5 and 10.20.16.5.2.

In addition, welded column splices that are subject to a calculated net tensile load effect determined using the load combinations stipulated by the Code including the amplified seismic load, shall satisfy both of the following requirements:

- (1) The available strength of partial-joint-penetration (PJP) groove welded joints, if used, shall be at least equal to 200 percent of the required strength.
- (2) The available strength for each flange splice shall be at least equal to $0.5R_yF_yA_f$ (LRFD) or $(0.5/1.5)R_yF_yA_f$ (ASD), as appropriate, where R_yF_y is the expected yield stress of the column material and A_f is the flange area of the smaller column connected.

Beveled transitions are not required when changes in thickness and width of flanges and webs occur in column splices where PJP groove welded joints are used.

Column web splices shall be either bolted or welded, or welded to one column and bolted to the other. In moment frames using bolted splices, plates or channels shall be used on both sides of the column web.

The centerline of column splices made with fillet welds or partial-joint-penetration groove welds shall be located 1.2 m or more away from the beam-to-column connections. When the column clear height between beam-to-column connections is less than 2.4 m, splices shall be at half the clear height.

10.20.8.4.2 Columns not part of the seismic load resisting system

Splices of columns that are not a part of the SLRS shall satisfy the following:

- (1) The splices shall be located 1.2 m or more away from the beam-to-column connections. When the column clear height between beam-to-column connections is less than 2.4 m, splices shall be at half the clear height.
- (2) The required shear strength of column splices with respect to both orthogonal axes of the column shall be M_{pc}/H (LRFD) or $M_{pc}/(1.5H)$ (ASD), as appropriate, where M_{pc} is the lesser nominal plastic flexural strength of the column sections for the direction in question, and H is the story height.

10.20.8.5 Column bases

The required strength of column bases shall be calculated in accordance with Sections 10.20.8.5.1, 10.20.8.5.2, and 10.20.8.5.3. The available strength of anchor rods shall be determined in accordance with Specification Sec 10.10.3.

The available strength of concrete elements at the column base, including anchor rod embedment and reinforcing steel, shall be in accordance with Appendix D Chapter 6 Part 6 of this Code.

Exception:

The special requirements in Appendix D Chapter 6 Part 6, for “regions of moderate or high seismic risk, or for structures assigned to intermediate or high seismic performance or design categories” need not be applied.

Table 6.10.18: Limiting Width-Thickness Ratios for Compression Elements

	Description of Element	Width-Thickness Ratio	Limiting Width- Thickness Ratios
			λ_{ps} (Seismically Compact)
Unstiffened Elements	Flexure in flanges of rolled or built-up I-shaped sections ^{[a], [c], [e], [g], [h]}	b/t	$0.30\sqrt{(E/F_y)}$
	Uniform compression in flanges of rolled or built-up I-shaped sections ^{[b], [h]}	b/t	$0.30\sqrt{(E/F_y)}$
	Uniform compression in flanges of rolled or built-up I-shaped sections ^[d]	b/t	$0.38\sqrt{(E/F_y)}$

	Description of Element	Width- Thickn ess Ratio	Limiting Width- Thickness Ratios
			λ_{ps} (Seismically Compact)
Unstiffened Elements	Uniform compression in flanges of channels, outstanding legs of pairs of angles in continuous contact, and braces ^[c] , ^[g]	b/t	$0.30\sqrt{(E/F_y)}$
	Uniform compression in flanges of H-pile sections	b/t	$0.45\sqrt{(E/F_y)}$
	Flat bars ^[f]	b/t	2.5
	Uniform compression in legs of single angles, legs of double angle members with separators, or flanges of tees ^[g]	b/t	$0.30\sqrt{(E/F_y)}$
	Uniform compression in stems of tees ^[g]	b/t	$0.30\sqrt{(E/F_y)}$
Stiffened Elements	Webs in flexural compression in beams in SMF, Sec 10.20.9, unless noted otherwise	h/t_w	$2.45\sqrt{(E/F_y)}$
	Webs in flexural compression or combined flexure and axial compression ^[a] , ^[c] , ^[g] , ^[h] , ^[i] , ^[j]	h/t_w	For $C_a > 0.125$ [k] $3.14\sqrt{(E/F_y)} (1 - 1.54C_a)$ For $C_a \leq 0.125$ [k] $1.12\sqrt{(E/F_y)} (2.33 - C_a)$ $\geq 1.49\sqrt{E/F_y}$
	Round HSS in axial and/or flexural compression ^[c] , ^[g]	D/t	$0.044\sqrt{(E/F_y)}$
	Rectangular HSS in axial and/or flexural compression ^[c] , ^[g]	b/t or h/t_w	$0.64\sqrt{(E/F_y)}$
	Webs of H-Pile sections	h/t_w	$0.94\sqrt{(E/F_y)}$

Notes:

- [a] Required of beams in SMF, Sec 10.20.9 and SPSW, Sec 10.20.17
- [b] Required of columns in SMF, Sec 10.20.9, unless the ratios from Eq. 6.10.300 are greater than 2.0 where it is permitted to use λ_p in Specification Table 6.10.1
- [c] Required for braces and columns in SCBF, Sec 10.20.13 and braces in OCBF, Sec 10.20.14
- [d] it is permitted to use λ_p in Specification Table 6.10.1 for columns in STMF, Sec 10.20.12 and columns in EBF, Sec 10.20.15
- [e] Required for link in EBF, Sec 10.20.15, except it is permitted to use λ_p in Table 6.10.1 of the Specification for flanges of links of length $1.6 M_p/V_p$ or less, where M_p and V_p are defined in Sec 10.20.15
- [f] Diagonal web members within the special segment of STMF, Sec 10.20.12
- [g] Chord members of STMF, Sec 10.20.12
- [h] Required for beams and columns in BRBF, Sec 10.20.16
- [i] Required for columns in SPSW, Sec 10.20.17
- [j] For columns in STMF, Sec 10.20.15; or EBF webs of links of length $1.6 M_p/V_p$ or less, it is permitted to use following for λ_p
 For $C_a \leq 0.125$, $\lambda_p = 3.76 \sqrt{E/F_y} (1 - 275C_a)$
 For $C_a > 0.125$, $\lambda_p = 1.12 \sqrt{E/F_y} (2.33 - C_a) \geq 1.49 \sqrt{E/F_y}$
- [k] For LFRD, $C_a = P_u/\phi_b P_y$; For ASD, $C_a = \Omega_b P_a/P_y$

Where,

P_a = required compressive strength (ASD), N

P_u = required compressive strength (LRFD), N

P_y = axial yield strength, N

$\phi_b = 0.90$

$\Omega_b = 1.67$

10.20.8.5.1 Required axial strength

The required axial strength of column bases, including their attachment to the foundation, shall be the summation of the vertical components of the required strengths of the steel elements that are connected to the column base.

10.20.8.5.2 Required shear strength

The required shear strength of column bases, including their attachments to the foundations, shall be the summation of the horizontal component of the required strengths of the steel elements that are connected to the column base as follows:

- (1) For diagonal bracing, the horizontal component shall be determined from the required strength of bracing connections for the seismic load resisting system (SLRS).
- (2) For columns, the horizontal component shall be at least equal to the lesser of the following:
 - (a) $2R_y F_y Z_x / H$ (LRFD) or $(2/1.5)R_y F_y Z_x / H$ (ASD), as appropriate, of the column

Where, H = height of story, which may be taken as the distance between the centerline of floor framing at each of the levels above and below, or the distance between the top of floor slabs at each of the levels above and below, mm.

- (b) The shear calculated using the load combinations of the applicable building Code, including the amplified seismic load.

10.20.8.5.3 Required flexural strength

The required flexural strength of column bases, including their attachment to the foundation, shall be the summation of the required strengths of the steel elements that are connected to the column base as follows:

- (1) For diagonal bracing, the required flexural strength shall be at least equal to the required strength of bracing connections for the SLRS.
- (2) For columns, the required flexural strength shall be at least equal to the lesser of the following:
 - (a) $1.1R_y F_y Z$ (LRFD) or $(1.1/1.5)R_y F_y Z$ (ASD), as appropriate, of the column
or
 - (b) moment calculated using the load combinations of the Code, including the amplified seismic load.

10.20.8.6 H-piles

10.20.8.6.1 Design of H-piles

Design of H-piles shall comply with the provisions of the Specification regarding design of members subjected to combined loads. H-piles shall meet the requirements of Sec 10.20.8.2.2.

10.20.8.6.2 Battered H-piles

If battered (sloped) and vertical piles are used in a pile group, the vertical piles shall be designed to support the combined effects of the dead and live loads without the participation of the battered piles.

10.20.8.6.3 Tension in H-piles

Tension in each pile shall be transferred to the pile cap by mechanical means such as shear keys, reinforcing bars or studs welded to the embedded portion of the pile. Directly below the bottom of the pile cap, each pile shall be free of attachments and welds for a length at least equal to the depth of the pile cross-section.

10.20.9 Special Moment Frames (SMF)

10.20.9.1 Scope

Special moment frames (SMF) are expected to withstand significant inelastic deformations when subjected to the forces resulting from the motions of the design earthquake. SMF shall satisfy the requirements in this Section.

10.20.9.2 Beam-to-column connections

10.20.9.2.1 Requirements

Beam-to-column connections used in the seismic load resisting system (SLRS) shall satisfy the following three requirements:

- (1) The connection shall be capable of sustaining an interstory drift angle of at least 0.04 radians.
- (2) The measured flexural resistance of the connection, determined at the column face, shall equal at least $0.80M_p$ of the connected beam at an interstory drift angle of 0.04 radians.

- (3) The required shear strength of the connection shall be determined using the following quantity for the earthquake load effect E :

$$E = 2[1.1R_yM_p]/L_h \quad (6.10.298)$$

Where,

R_y = ratio of the expected yield stress to the specified minimum yield stress, F_y

M_p = nominal plastic flexural strength, (N-mm)

L_h = distance between plastic hinge locations, (mm)

When E as in Eq. 6.10.298 is used in ASD load combinations that are additive with other transient loads and that are based on Chapter 2 Part 6, the 0.75 combination factor for transient loads shall not be applied to E .

Connections that accommodate the required interstory drift angle within the connection elements and provide the measured flexural resistance and shear strengths specified above are permitted. In addition to satisfying the requirements noted above, the design shall demonstrate that any additional drift due to connection deformation can be accommodated by the structure. The design shall include analysis for stability effects of the overall frame, including second-order effects.

10.20.9.2.2 Conformance demonstration

Beam-to-column connections used in the SLRS shall satisfy the requirements of Sec 10.20.9.2.1 by one of the following:

- (a) Use of SMF connections designed in accordance with ANSI/AISC 358.
- (b) Use of a connection prequalified for SMF in accordance with Appendix N.
- (c) Provision of qualifying cyclic test results in accordance with Appendix Q.

Results of at least two cyclic connection tests shall be provided and are permitted to be based on one of the following:

- (i) Tests reported in the research literature or documented tests performed for other projects that represent the project conditions, within the limits specified in Appendix Q.
- (ii) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes, within the limits specified in Appendix Q.

10.20.9.2.3 Welds

Unless otherwise designated by ANSI/AISC 358, or otherwise determined in a connection prequalification in accordance with Appendix N, or as determined in a program of qualification testing in accordance with Appendix Q, complete-joint-penetration groove welds of beam flanges, shear plates, and beam webs to columns shall be demand critical welds as described in Sec 10.20.7.3.2.

10.20.9.2.4 Protected zones

The region at each end of the beam subject to inelastic straining shall be designated as a protected zone, and shall meet the requirements of Sec 10.20.7.4. The extent of the protected zone shall be as designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix N, or as determined in a program of qualification testing in accordance with Appendix Q.

10.20.9.3 Panel zone of beam-to-column connections (beam web parallel to column web)

10.20.9.3.1 Shear strength

The required thickness of the panel zone shall be determined in accordance with the method used in proportioning the panel zone of the tested or prequalified connection. As a minimum, the required shear strength of the panel zone shall be determined from the summation of the moments at the column faces as determined by projecting the expected moments at the plastic hinge points to the column faces. The design shear strength shall be $\phi_v R_v$ and the allowable shear strength shall be R_v/Ω_v .

Where, $\phi_v = 1.0$ (LRFD) $\Omega_v = 1.50$ (ASD)

And, the nominal shear strength, R_v , according to the limit state of shear yielding, is determined as specified in Specification Sec 10.10.10.6.

10.20.9.3.2 Panel zone thickness

The individual thicknesses, t , of column webs and doubler plates, if used, shall conform to the following requirement:

$$t \geq (d_z + w_z)/90 \quad (6.10.299)$$

Where,

t = thickness of column web or doubler plate, mm

d_z = panel zone depth between continuity plates, mm

w_z = panel zone width between column flanges, mm

Alternatively, when local buckling of the column web and doubler plate is prevented by using plug welds joining them, the total panel zone thickness shall satisfy Eq. 6.10.299.

10.20.9.3.3 Panel zone doubler plates

Doubler plates shall be welded to the column flanges using either a complete-joint-penetration groove-welded or fillet-welded joint that develops the available shear strength of the full doubler plate thickness. When doubler plates are placed against the column web, they shall be welded across the top and bottom edges to develop the proportion of the total force that is transmitted to the doubler plate. When doubler plates are placed away from the column web, they shall be placed symmetrically in pairs and welded to continuity plates to develop the proportion of the total force that is transmitted to the doubler plate.

10.20.9.4 Beam and column limitations

The requirements of Sec 10.20.8.1 shall be satisfied, in addition to the following.

10.20.9.4.1 Width-thickness limitations

Beam and column members shall meet the requirements of Sec 10.20.8.2.2, unless otherwise qualified by tests.

10.20.9.4.2 Beam flanges

Abrupt changes in beam flange area are not permitted in plastic hinge regions. The drilling of flange holes or trimming of beam flange width is permitted if testing or qualification demonstrates that the resulting configuration can develop stable plastic hinges. The configuration shall be consistent with a prequalified connection designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix N, or in a program of qualification testing in accordance with Appendix Q.

10.20.9.5 Continuity plates

Continuity plates shall be consistent with the prequalified connection designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix N, or as determined in a program of qualification testing in accordance with Appendix Q.

10.20.9.6 Column-beam moment ratio

The following relationship shall be satisfied at beam-to-column connections:

$$\frac{\sum M_{pc}^*}{\sum M_{pb}^*} > 1.0 \quad (6.10.300)$$

$\sum M_{pc}^*$ = the sum of the moments in the column above and below the joint at the intersection of the beam and column centerlines. $\sum M_{pc}^*$ is determined by summing the projections of the nominal flexural strengths of the columns (including haunches where used) above and below the joint to the beam centerline with a reduction for the axial force in the column. It is permitted to take $\sum M_{pc}^* = \sum Z_c (F_{yc} - P_{uc}/A_g)$ (LRFD) or $\sum Z_c (F_{yc}/1.5 - P_{ac}/A_g)$ (ASD), as appropriate. When the centerlines of opposing beams in the same joint do not coincide, the mid-line between centerlines shall be used.

$\sum M_{pb}^*$ = the sum of the moments in the beams at the intersection of the beam and column centerlines. $\sum M_{pb}^*$ is determined by summing the projections of the expected flexural strengths of the beams at the plastic hinge locations to the column centerline. It is permitted to take $\sum M_{pb}^* = \sum (1.1R_y F_{yb} Z_b + M_{uv})$ (LRFD) or $\sum M_{pb}^* = \sum [(1.1/1.5)R_y F_{yb} Z_b + M_{av}]$ (ASD), as appropriate. Alternatively, it is permitted to determine $\sum M_{pb}^*$ consistent with a prequalified connection design as designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix N, or in a program of qualification testing in accordance with Appendix Q. When connections with reduced beam sections are used, it is permitted to take $\sum M_{pb}^* = \sum (1.1R_y F_{yb} Z_{RBS} + M_{uv})$ (LRFD) or $\sum M_{pb}^* = \sum [(1.1/1.5)R_y F_{yb} Z_{RBS} + M_{av}]$ (ASD), as appropriate.

Where,

A_g = gross area of column, mm

F_{yc} = specified minimum yield stress of column, MPa

M_{av} = the additional moment due to shear amplification from the location of the plastic hinge to the column centerline, based on ASD load combinations, N-mm.

M_{uv} = the additional moment due to shear amplification from the location of the plastic hinge to the column centerline, based on LRFD load combinations, N-mm

P_{ac} = required compressive strength using ASD load combinations, (positive number) N.

P_{uc} = required compressive strength using LRFD load combinations, (positive number) N

Z_b = plastic section modulus of the beam, mm³

Z_c = plastic section modulus of the column, mm³

Z_{RBS} = minimum plastic section modulus at the reduced beam section, mm³

Exception:

This requirement does not apply if either of the following two conditions is satisfied:

- (a) Columns with $P_{rc} < 0.3P_c$ for all load combinations other than those determined using the amplified seismic load that satisfy either of the following:
 - (i) Columns used in a one-story building or the top story of a multistory building.
 - (ii) Columns where: (1) the sum of the available shear strengths of all exempted columns in the story is less than 20 percent of the sum of the available shear strengths of all moment frame columns in the story acting in the same direction; and (2) the sum of the available shear strengths of all exempted columns on each moment frame column line within that story is less than 33 percent of the available shear strength of all moment frame columns on that column line. For the purpose of this exception, a column line is defined as a single line of columns or parallel lines of columns located within 10 percent of the plan dimension perpendicular to the line of columns.

Where,

For design according to Specification Sec 10.2.3.3 (LRFD),

$$P_c = F_{yc}A_g, \text{ N}$$

$$P_{rc} = P_{uc}, \text{ required compressive strength, using LRFD load combinations, N}$$

For design according to Specification Sec 10.2.3.4 (ASD),

$$P_c = F_{yc}A_g/1.5, \text{ N}$$

$$P_{rc} = P_{ac}, \text{ required compressive strength, using ASD load combinations, N}$$

- (b) Columns in any story that has a ratio of available shear strength to required shear strength that is 50 percent greater than the story above.

10.20.9.7 Lateral bracing at beam-to-column connections

10.20.9.7.1 Braced connections

Column flanges at beam-to-column connections require lateral bracing only at the level of the top flanges of the beams, when the webs of the beams and column are co-planar, and a column is shown to remain elastic outside of the panel zone. It shall be permitted to assume that the column remains elastic when the ratio calculated using Eq. 6.10.300 is greater than 2.0.

When a column cannot be shown to remain elastic outside of the panel zone, following requirements shall apply:

The column flanges shall be laterally braced at the levels of both the top and bottom beam flanges. Lateral bracing shall be either direct or indirect.

Each column-flange lateral brace shall be designed for a required strength that is equal to 2 percent of the available beam flange strength $F_y b_f t_{bf}$ (LRFD) or $F_y b_f t_{bf}/1.5$ (ASD), as appropriate.

10.20.9.7.2 Unbraced connections

A column containing a beam-to-column connection with no lateral bracing transverse to the seismic frame at the connection shall be designed using the distance between adjacent lateral braces as the column height for buckling transverse to the seismic frame and shall conform to Specification Sec 10.8, except that:

- (1) The required column strength shall be determined from the appropriate load combinations, except that E shall be taken as the lesser of:
 - (a) The amplified seismic load.
 - (b) 125 percent of the frame available strength based upon either the beam available flexural strength or panel zone available shear strength.

- (2) The slenderness L/r for the column shall not exceed 60.
- (3) The column required flexural strength transverse to the seismic frame shall include that moment caused by the application of the beam flange force specified in Sec 10.20.9.7.1.(2) in addition to the second-order moment due to the resulting column flange displacement.

10.20.9.8 Lateral bracing of beams

Both flanges of beams shall be laterally braced, with a maximum spacing of $L_b = 0.086r_y E / F_y$. Braces shall meet the provisions of Eq. 6.10.291 and 6.10.292 of Sec 10.19, where $M_r = M_u = R_y Z F_y$ (LRFD) or $M_r = M_a = R_y Z F_y / 1.5$ (ASD), as appropriate, of the beam and $C_d = 1.0$.

In addition, lateral braces shall be placed near concentrated forces, changes in cross-section, and other locations where analysis indicates that a plastic hinge will form during inelastic deformations of the SMF. The placement of lateral bracing shall be consistent with that documented for a prequalified connection designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix N, or in a program of qualification testing in accordance with Appendix Q.

The required strength of lateral bracing provided adjacent to plastic hinges shall be $P_u = 0.06M_u/h_o$ (LRFD) or $P_a = 0.06M_a/h_o$ (ASD), as appropriate, where h_o is the distance between flange centroids; and the required stiffness shall meet the provisions of Eq. 6.10.292 of Sec 10.19.

10.20.9.9 Column splices

Column splices shall comply with the requirements of Sec 10.20.8.4.1. Where groove welds are used to make the splice, they shall be complete-joint-penetration groove welds that meet the requirements of Sec 10.20.7.3.2. Weld tabs shall be removed. When column splices are not made with groove welds, they shall have a required flexural strength that is at least equal to $R_y F_y Z_x$ (LRFD) or $R_y F_y Z_x / 1.5$ (ASD), as appropriate, of the smaller column. The required shear strength of column web splices shall be at least equal to $\sum M_{pc} / H$ (LRFD) or $\sum M_{pc} / (1.5H)$ (ASD), as appropriate, where $\sum M_{pc}$ is the sum of the nominal plastic flexural strengths of the columns above and below the splice.

Exception:

The required strength of the column splice considering appropriate stress concentration factors or fracture mechanics stress intensity factors need not exceed that determined by inelastic analyses.

10.20.10 Intermediate Moment Frames (IMF)

10.20.10.1 Scope

Intermediate moment frames (IMF) are expected to withstand limited inelastic deformations in their members and connections when subjected to the forces resulting from the motions of the design earthquake. IMF shall meet the requirements in this Section.

10.20.10.2 Beam-to-column connections

10.20.10.2.1 Requirements

Beam-to-column connections used in the seismic load resisting system (SLRS) shall satisfy the requirements of Sec 10.20.9.2.1, with the following exceptions:

- (1) The required interstory drift angle shall be a minimum of 0.02 radian.
- (2) The required strength in shear shall be determined as specified in Sec 10.20.9.2.1, except that a lesser value of V_u or V_a , as appropriate, is permitted if justified by analysis. The required shear strength need not exceed the shear resulting from the application of appropriate load combinations using the amplified seismic load.

10.20.10.2.2 Conformance demonstration

Conformance demonstration shall be as described in Sec 10.20.9.2.2 to satisfy the requirements of Sec 10.20.10.2.1 for IMF, except that a connection prequalified for IMF in accordance with ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix N, or as determined in a program of qualification testing in accordance with Appendix Q.

10.20.10.2.3 Welds

Unless otherwise designated by ANSI/AISC 358, or otherwise determined in a connection prequalification in accordance with Appendix N, or as determined in a program of qualification testing in accordance with Appendix Q, complete joint penetration groove welds of beam flanges, shear plates, and beam webs to columns shall be demand critical welds as described in Sec 10.20.7.3.2.

10.20.10.2.4 Protected zone

The region at each end of the beam subject to inelastic straining shall be treated as a protected zone, and shall meet the requirements of Sec 10.20.7.4. The extent of the protected zone shall be as designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix N, or as determined in a program of qualification testing in accordance with Appendix Q.

10.20.10.3 Panel zone of beam-to-column connections (beam web parallel to column web)

No additional requirements beyond the specification.

10.20.10.4 Beam and column limitations

The requirements of Sec 10.20.8.1 shall be satisfied, in addition to the following.

10.20.10.4.1 Width-thickness limitations

Beam and column members shall meet the requirements of Sec 10.20.8.2.1, unless otherwise qualified by tests.

10.20.10.4.2 Beam flanges

Abrupt changes in beam flange area are not permitted in plastic hinge regions. Drilling of flange holes or trimming of beam flange width is permitted if testing or qualification demonstrates that the resulting configuration can develop stable plastic hinges. The configuration shall be consistent with a prequalified connection designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix N, or in a program of qualification testing in accordance with Appendix Q.

10.20.10.5 Continuity plates

Continuity plates shall be provided to be consistent with the prequalified connections designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix N, or as determined in a program of qualification testing in accordance with Appendix Q.

10.20.10.6 Column-beam moment ratio

No additional requirements beyond the Specification.

10.20.10.7 Lateral bracing at beam-to-column connections

No additional requirements beyond the Specification.

10.20.10.8 Lateral bracing of beams

Both flanges shall be laterally braced directly or indirectly. The unbraced length between lateral braces shall not exceed $0.17r_y E / F_y$. Braces shall meet the provisions of Equations 6.10.291 and 6.10.292 of Sec 10.19, where $M_r = M_u = R_y Z F_y$ (LRFD) or $M_r = M_a = R_y Z F_y / 1.5$ (ASD), as appropriate, of the beam, and $C_d = 1.0$.

In addition, lateral braces shall be placed near concentrated loads, changes in cross-section and other locations where analysis indicates that a plastic hinge will form during inelastic deformations of the IMF. Where the design is based upon assemblies tested in accordance with Appendix Q, the placement of lateral bracing for the beams shall be consistent with that used in the tests or as required for prequalification in Appendix N. The required strength of lateral bracing provided adjacent to plastic hinges shall be $P_u = 0.06M_u / h_o$ (LRFD) or $P_a = 0.06M_a / h_o$ (ASD), as appropriate, where h_o = distance between flange centroids; and the required stiffness shall meet the provisions of Eq. 6.10.292 of Sec 10.19.

10.20.10.9 Column splices

Column splices shall comply with the requirements of Sec 10.20.8.4.1. Where groove welds are used to make the splice, they shall be complete-joint-penetration groove welds that meet the requirements of Sec 10.20.7.3.2.

10.20.11 Ordinary Moment Frames (OMF)**10.20.11.1 Scope**

Ordinary moment frames (OMF) are expected to withstand minimal inelastic deformations in their members and connections when subjected to the forces resulting from the motions of the design earthquake. OMF shall meet the requirements of this Section. Connections in conformance with Sections 10.20.9.2.1 and 10.20.9.5 or Sections 10.20.10.2.1 and 10.20.10.5 shall be permitted for use in OMF without meeting the requirements of Sections 10.20.11.2.1, 10.20.11.2.3, and 10.20.11.5

10.20.11.2 Beam-to-column connections

Beam-to-column connections shall be made with welds and/or high-strength bolts. Connections are permitted to be fully restrained (FR) or partially restrained (PR) moment connections as follows.

10.20.11.2.1 Requirements for FR moment connections

FR moment connections that are part of the seismic load resisting system (SLRS) shall be designed for a required flexural strength that is equal to $1.1R_yM_p$ (LRFD) or $(1.1/1.5)R_yM_p$ (ASD), as appropriate, of the beam or girder, or the maximum moment that can be developed by the system, whichever is less.

FR connections shall meet the following requirements.

- (1) Where steel backing is used in connections with complete-joint-penetration (CJP) beam flange groove welds, steel backing and tabs shall be removed, except that top-flange backing attached to the column by a continuous fillet weld on the edge below the CJP groove weld need not be removed. Removal of steel backing and tabs shall be as follows:
 - (i) Following the removal of backing, the root pass shall be back gouged to sound weld metal and back welded with a reinforcing fillet. The reinforcing fillet shall have a minimum leg size of 8 mm.
 - (ii) Weld tab removal shall extend to within 3 mm of the base metal surface, except at continuity plates where removal to within 6 mm of the plate edge is acceptable. Edges of the weld tab shall be finished to a surface roughness value of $13\ \mu\text{m}$ or better. Grinding to a flush condition is not required. Gouges and notches are not permitted. The transitional slope of any area where gouges and notches have been removed shall not exceed 1:5. Material removed by grinding that extends more than 2 mm below the surface of the base metal shall be filled with weld metal. The contour of the weld at the ends shall provide a smooth transition, free of notches and sharp corners.
- (2) Where weld access holes are provided, they shall be as shown in Figure 6.10.3. The weld access hole shall have a surface roughness value not to exceed $13\ \mu\text{m}$, and shall be free of notches and gouges. Notches and gouges shall be repaired as required by the engineer of record. Weld access holes are prohibited in the beam web adjacent to the end-plate in bolted moment end-plate connections.
- (3) The required strength of double-sided partial-joint-penetration groove welds and double-sided fillet welds that resist tensile forces in connections shall be $1.1R_yF_yA_g$ (LRFD) or $(1.1/1.5)R_yF_yA_g$ (ASD), as appropriate, of the connected element or part. Single-sided partial-joint-penetration groove welds and single-sided fillet welds shall not be used to resist tensile forces in the connections.

- (4) For FR moment connections, the required shear strength, V_u or V_a , as appropriate, of the connection shall be determined using the following quantity for the earthquake load effect E :

$$E = 2(1.1R_y M_p)/L_b \quad (6.10.301)$$

Where this E is used in ASD load combinations that are additive with other transient loads and that are based on Chapter 2 Part 6, the 0.75 combination factor for transient loads shall not be applied to E .

Alternatively, a lesser value of V_u or V_a is permitted if justified by analysis. The required shear strength need not exceed the shear resulting from the application of appropriate load combinations in the Code using the amplified seismic load.

10.20.11.2.2 Requirements for PR moment connections

PR moment connections are permitted when the following requirements are met:

- (1) Such connections shall be designed for the required strength as specified in Sec 10.20.11.2.1 above.
- (2) The nominal flexural strength of the connection, M_n , shall be no less than 50 percent of M_p of the connected beam or column, whichever is less.
- (3) The stiffness and strength of the PR moment connections shall be considered in the design, including the effect on overall frame stability.
- (4) For PR moment connections, V_u or V_a , as appropriate, shall be determined from the load combination above plus the shear resulting from the maximum end moment that the connection is capable of resisting.

10.20.11.2.3 Welds

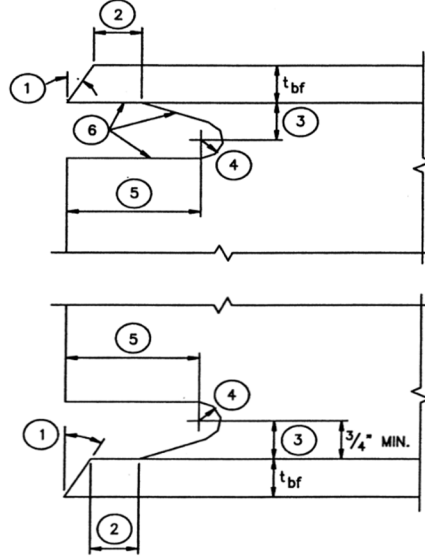
Complete-joint-penetration groove welds of beam flanges, shear plates, and beam webs to columns shall be demand critical welds as described in Sec 10.20.7.3.2.

10.20.11.3 Panel zone of beam-to-column connections (beam web parallel to column web)

No additional requirements beyond the Specification.

10.20.11.4 Beam and column limitations

No requirements beyond Sec 10.20.8.1.



Notes:

1. Bevel as required for selected groove weld.
2. Larger of t_{bf} or 13 mm (plus $\frac{1}{2} t_{bf}$, or minus $\frac{1}{4} t_{bf}$)
3. $\frac{3}{4} t_{bf}$ to t_{bf} , 19 mm minimum (± 6 mm)
4. 10 mm minimum radius (plus not limited, minus 0)
5. $3 t_{bf}$ (± 13 mm)

Tolerances shall not accumulate to the extent that the angle of the access hole cut to the flange surface exceeds 25° .

Figure 6.10.3. Weld access hole detail (FEMA 350)

10.20.11.5 Continuity plates

When FR moment connections are made by means of welds of beam flanges or beam-flange connection plates directly to column flanges, continuity plates shall be provided in accordance with Sec J10 of the Specification. Continuity plates shall also be required when:

$$t_{cf} < 0.54(b_f t_{bf} F_{yb} / F_{yc})^{1/2}$$

Or, when,

$$t_{cf} < b_f / 6$$

Where continuity plates are required, the thickness of the plates shall be determined as follows:

- (a) For one-sided connections, continuity plate thickness shall be at least one half of the thickness of the beam flange.
- (b) For two-sided connections the continuity plates shall be at least equal in thickness to the thicker of the beam flanges.

The welded joints of the continuity plates to the column flanges shall be made with either complete-joint-penetration groove welds, two-sided partial-joint-penetration groove welds combined with reinforcing fillet welds, or two-sided fillet welds. The required strength of these joints shall not be less than the available strength of the contact area of the plate with the column flange. The required strength of the welded joints of the continuity plates to the column web shall be the least of the following:

- (a) The sum of the available strengths at the connections of the continuity plate to the column flanges.
- (b) The available shear strength of the contact area of the plate with the column web.
- (c) The weld available strength that develops the available shear strength of the column panel zone.
- (d) The actual force transmitted by the stiffener.

10.20.11.6 Column-beam moment ratio

No requirements.

10.20.11.7 Lateral bracing at beam-to-column connections

No additional requirements beyond the Specification.

10.20.11.8 Lateral bracing of beams

No additional requirements beyond the Specification.

10.20.11.9 Column splices

Column splices shall comply with the requirements of Sec 10.20.8.4.1.

10.20.12 Special Truss Moment Frames (STMF)

10.20.12.1 Scope

Special truss moment frames (STMF) are expected to withstand significant inelastic deformation within a specially designed segment of the truss when subjected to the forces from the motions of the design earthquake. STMF shall be limited to span lengths between

columns not to exceed 20 m and overall depth not to exceed 1.8 m. The columns and truss segments outside of the special segments shall be designed to remain elastic under the forces that can be generated by the fully yielded and strain-hardened special segment. STMF shall meet the requirements in this Section.

10.20.12.2 Special segment

Each horizontal truss that is part of the seismic load resisting system (SLRS) shall have a special segment that is located between the quarter points of the span of the truss. The length of the special segment shall be between 0.1 and 0.5 times the truss span length. The length-to-depth ratio of any panel in the special segment shall neither exceed 1.5 nor be less than 0.67.

Panels within a special segment shall either be all Vierendeel panels or all X-braced panels; neither a combination thereof nor the use of other truss diagonal configurations is permitted. Where diagonal members are used in the special segment, they shall be arranged in an X pattern separated by vertical members. Such diagonal members shall be interconnected at points where they cross. The interconnection shall have a required strength equal to 0.25 times the nominal tensile strength of the diagonal member. Bolted connections shall not be used for web members within the special segment. Diagonal web members within the special segment shall be made of flat bars of identical sections.

Splicing of chord members is not permitted within the special segment, nor within one-half the panel length from the ends of the special segment. The required axial strength of the diagonal web members in the special segment due to dead and live loads within the special segment shall not exceed $0.03F_yA_g$ (LRFD) or $(0.03/1.5)F_yA_g$ (ASD), as appropriate. The special segment shall be a protected zone meeting the requirements of Sec 10.20.7.4.

10.20.12.3 Strength of special segment members

The available shear strength of the special segment shall be calculated as the sum of the available shear strength of the chord members through flexure, and the shear strength corresponding to the available tensile strength and 0.3 times the available compressive strength of the diagonal members, when they are used. The top and bottom chord members in the special segment shall be made of identical sections and shall provide at least 25 percent of the required vertical shear strength. The required axial strength in the chord members, determined according to the limit state of tensile yielding, shall not exceed 0.45 times ϕP_n (LRFD) or P_n/Ω (ASD), as appropriate,

$$\phi = 0.90 \text{ (LRFD)}$$

$$\Omega = 1.67 \text{ (ASD)}$$

Where,

$$P_n = F_y A_g$$

The end connection of diagonal web members in the special segment shall have a required strength that is at least equal to the expected yield strength, in tension, of the web member, $R_y F_y A_g$ (LRFD) or $R_y F_y A_g / 1.5$ (ASD), as appropriate.

10.20.12.4 Strength of non-special segment members

Members and connections of STMF, except those in the special segment specified in Sec 10.20.12.2, shall have a required strength based on the appropriate load combinations in the Code, replacing the earthquake load term E with the lateral loads necessary to develop the expected vertical shear strength of the special segment V_{ne} (LRFD) or $V_{ne} / 1.5$ (ASD), as appropriate, at mid-length, given as:

$$V_{ne} = \frac{3.75 R_y M_{nc}}{L_s} + 0.075 EI \frac{L - L_s}{L_s^3} + R_y (P_{nt} + 0.3 P_{nc}) \sin \alpha \quad (6.10.302)$$

Where,

M_{nc} = nominal flexural strength of a chord member of the special segment,
N-mm

EI = flexural elastic stiffness of a chord member of the special segment, N-mm²

L = span length of the truss, mm

L_s = length of the special segment, mm

P_{nt} = nominal tensile strength of a diagonal member of the special segment, N

P_{nc} = nominal compressive strength of a diagonal member of the special segment,
N

α = angle of diagonal members with the horizontal

10.20.12.5 Width-thickness limitations

Chord members and diagonal web members within the special segment shall meet the requirements of Sec 10.20.8.2.2.

10.20.12.6 Lateral bracing

The top and bottom chords of the trusses shall be laterally braced at the ends of the special segment, and at intervals not to exceed L_p according to Specification Sec 10.6 along the entire length of the truss. The required strength of each lateral brace at the ends of and within the special segment shall be

$$P_u = 0.06R_yP_{nc} \text{ (LRFD) or } P_a = (0.06/1.5)R_yP_{nc} \text{ (ASD), as appropriate.}$$

Where, P_{nc} is the nominal compressive strength of the special segment chord member. Lateral braces outside of the special segment shall have a required strength of

$$P_u = 0.02R_yP_{nc} \text{ (LRFD) or } P_a = (0.02/1.5)R_yP_{nc} \text{ (ASD), as appropriate.}$$

The required brace stiffness shall meet the provisions of Eq. 6.10.288 of Sec 10.19.

Where, $P_r = P_u = R_yP_{nc}$ (LRFD) or $P_r = P_a = R_yP_{nc}/1.5$ (ASD), as appropriate.

10.20.13 Special Concentrically Braced Frames (SCBF)

10.20.13.1 Scope

Special concentrically braced frames (SCBF) are expected to withstand significant inelastic deformations when subjected to the forces resulting from the motions of the design earthquake. SCBF shall meet the requirements in this Section.

10.20.13.2 Members

10.20.13.2.1 Slenderness

Bracing members shall have $KI/r \leq 4\sqrt{(E/F_y)}$.

Exception :

Braces with $4\sqrt{(E/F_y)} < KI/r \leq 200$ are permitted in frames in which the available strength of the column is at least equal to the maximum load transferred to the column considering R_y (LRFD) or $(1/1.5)R_y$ (ASD), as appropriate, times the nominal strengths of the connecting brace elements of the building. Column forces need not exceed those determined by inelastic analysis, nor the maximum load effects that can be developed by the system.

10.20.13.2.2 Required strength

Where the effective net area of bracing members is less than the gross area, the required tensile strength of the brace based upon the limit state of fracture in the net section shall be greater than the lesser of the following:

- (a) The expected yield strength, in tension, of the bracing member, determined as $R_y F_y A_g$ (LRFD) or $R_y F_y A_g / 1.5$ (ASD), as appropriate.
- (b) The maximum load effect, indicated by analysis that can be transferred to the brace by the system.

10.20.13.2.3 Lateral force distribution

Along any line of bracing, braces shall be deployed in alternate directions such that, for either direction of force parallel to the bracing, at least 30 percent but no more than 70 percent of the total horizontal force along that line is resisted by braces in tension, unless the available strength of each brace in compression is larger than the required strength resulting from the application of the appropriate load combinations stipulated by the Code including the amplified seismic load. For the purposes of this provision, a line of bracing is defined as a single line or parallel lines with a plan offset of 10 percent or less of building dimension perpendicular to line of bracing.

10.20.13.2.4 Width-thickness limitations

Column and brace members shall meet the requirements of Sec 10.20.8.2.2.

10.20.13.2.5 Built-up members

The spacing of stitches shall be such that the slenderness ratio l/r of individual elements between the stitches does not exceed 0.4 times the governing slenderness ratio of the built-up member.

The sum of the available shear strengths of the stitches shall equal or exceed the available tensile strength of each element. The spacing of stitches shall be uniform. Not less than two stitches shall be used in a built-up member. Bolted stitches shall not be located within the middle one-fourth of the clear brace length.

Exception:

Where the buckling of braces about their critical buckling axis does not cause shear in the stitches, the spacing of the stitches shall be such that the slenderness ratio l/r of the individual elements between the stitches does not exceed 0.75 times the governing slenderness ratio of the built-up member.

10.20.13.3 Required strength of bracing connections

10.20.13.3.1 Required tensile strength

The required tensile strength of bracing connections (including beam-to-column connections if part of the bracing system) shall be the lesser of the following:

- (a) The expected yield strength, in tension, of the bracing member, determined as $R_y F_y A_g$ (LRFD) or $R_y F_y A_g / 1.5$ (ASD), as appropriate.
- (b) The maximum load effect, indicated by analysis that can be transferred to the brace by the system.

10.20.13.3.2 Required flexural strength

The required flexural strength of bracing connections shall be equal to $1.1 R_y M_p$ (LRFD) or $(1.1/1.5) R_y M_p$ (ASD), as appropriate, of the brace about the critical buckling axis.

Exception:

Brace connections that meet the requirements of Sec 10.20.13.3.1 and can accommodate the inelastic rotations associated with brace post-buckling deformations need not meet this requirement.

10.20.13.3.3 Required compressive strength

Bracing connections shall be designed for a required compressive strength based on buckling limit states that is at least equal to $1.1 R_y P_n$ (LRFD) or $(1.1/1.5) R_y P_n$ (ASD), as appropriate, where P_n is the nominal compressive strength of the brace.

10.20.13.4 Special bracing configuration requirements

10.20.13.4.1 V-type and inverted-V-type bracing

V-type and inverted V-type SCBF shall meet the following requirements:

- (1) The required strength of beams intersected by braces, their connections, and supporting members shall be determined based on the load combinations of the Code assuming that the braces provide no support for dead and live loads. For load combinations that include earthquake effects, the earthquake effect, E , on the beam shall be determined as follows:
 - (a) The forces in all braces in tension shall be assumed to be equal to $R_y F_y A_g$.
 - (b) The forces in all adjoining braces in compression shall be assumed to be equal to $0.3P_n$.
- (2) Beams shall be continuous between columns. Both flanges of beams shall be laterally braced, with a maximum spacing of $L_b = L_{pd}$, as specified by Equations 6.10.270 and 6.10.271 of Sec 10.15. Lateral braces shall meet the provisions of Equations 6.10.291 and 6.10.292 of Sec 10.19, where $M_r = M_u = R_y Z F_y$ (LRFD) or $M_r = M_a = R_y Z F_y / 1.5$ (ASD), as appropriate, of the beam and $C_d = 1.0$.

As a minimum, one set of lateral braces is required at the point of intersection of the V-type (or inverted V-type) bracing, unless the beam has sufficient out-of-plane strength and stiffness to ensure stability between adjacent brace points.

10.20.13.4.2 K-type bracing

K-type braced frames are not permitted for SCBF.

10.20.13.5 Column splices

In addition to meeting the requirements in Sec 10.20.8.4, column splices in SCBF shall be designed to develop 50 percent of the lesser available flexural strength of the connected members. The required shear strength shall be $\sum M_{pc} / H$ (LRFD) or $\sum M_{pc} / 1.5H$ (ASD), as appropriate, where $\sum M_{pc}$ is the sum of the nominal plastic flexural strengths of the columns above and below the splice.

10.20.13.6 Protected zone

The protected zone of bracing members in SCBF shall include the center one-quarter of the brace length, and a zone adjacent to each connection equal to the brace depth in the plane of buckling. The protected zone of SCBF shall include elements that connect braces to beams and columns and shall satisfy the requirements of Sec 10.20.7.4.

10.20.14 Ordinary Concentrically Braced Frames (OCBF)**10.20.14.1 Scope**

Ordinary concentrically braced frames (OCBF) are expected to withstand limited inelastic deformations in their members and connections when subjected to the forces resulting from the motions of the design earthquake. OCBF shall meet the requirements in this Section. OCBF above the isolation system in seismically isolated structures shall meet the requirements of Sections 10.20.14.4 and 10.20.14.5 and need not meet the requirements of Sections 10.20.14.2 and 10.20.14.3.

10.20.14.2 Bracing members

Bracing members shall meet the requirements of Sec 10.20.8.2.2.

Exception:

HSS braces that are filled with concrete need not comply with this provision.

Bracing members in K, V, or inverted-V configurations shall have $KL/r \leq 4 \sqrt{(E/F_y)}$.

10.20.14.3 Special bracing configuration requirements

Beams in V-type and inverted V-type OCBF and columns in K-type OCBF shall be continuous at bracing connections away from the beam-column connection and shall meet the following requirements:

- (1) The required strength shall be determined based on the load combinations of the Code assuming that the braces provide no support of dead and live loads. For load combinations that include earthquake effects, the earthquake effect, E , on the member shall be determined as follows:
 - (a) The forces in braces in tension shall be assumed to be equal to $R_y F_y A_g$. For V-type and inverted V-type OCBF, the forces in braces in tension need not exceed the maximum force that can be developed by the system.
 - (b) The forces in braces in compression shall be assumed to be equal to $0.3P_n$.
- (2) Both flanges shall be laterally braced, with a maximum spacing of $L_b = L_{pd}$, as specified by Equations 6.10.270 and 6.10.271 of Sec 10.15.. Lateral braces shall meet the provisions of Equations 6.10.291 and 6.10.292 of Sec 10.19, where $M_r = M_u = R_y Z F_y$ (LRFD) or $M_r = M_a = R_y Z F_y / 1.5$ (ASD), as appropriate, of the beam

and $C_d = 1.0$. As a minimum, one set of lateral braces is required at the point of intersection of the bracing, unless the member has sufficient out-of-plane strength and stiffness to ensure stability between adjacent brace points.

10.20.14.4 Bracing connections

The required strength of bracing connections shall be determined as follows.

- (1) For the limit state of bolt slip, the required strength of bracing connections shall be that determined using the load combinations stipulated by the Code, not including the amplified *seismic load*.
- (2) For other limit states, the required strength of bracing connections is the *expected yield strength*, in tension, of the brace, determined as $R_y F_y A_g$ (LRFD) or $R_y F_y A_g / 1.5$ (ASD), as appropriate.

Exception:

The required strength of the brace connection need not exceed either of the following:

- (a) The maximum force that can be developed by the system
- (b) A load effect based upon using the amplified seismic load

10.20.14.5 OCBF above seismic isolation systems

10.20.14.5.1 Bracing members

Bracing members shall meet the requirements of Sec 10.20.8.2.2 and shall have

$$KL/r \leq 4\sqrt{(E/F_y)}.$$

10.20.14.5.2 K-type bracing

K-type braced frames are not permitted.

10.20.14.5.3 V-type and inverted-V-type bracing

Beams in V-type and inverted V-type bracing shall be continuous between columns.

10.20.15 Eccentrically Braced Frames (EBF)**10.20.15.1 Scope**

Eccentrically braced frames (EBFs) are expected to withstand significant inelastic deformations in the links when subjected to the forces resulting from the motions of the design earthquake. The diagonal braces, columns, and beam segments outside of the links shall be designed to remain essentially elastic under the maximum forces that can be generated by the fully yielded and strain-hardened links, except where permitted in this Section. In buildings exceeding five stories in height, the upper story of an EBF system is permitted to be designed as an OCBF or a SCBF and still be considered to be part of an EBF system for the purposes of determining system factors in the Code. EBF shall meet the requirements in this Section.

10.20.15.2 Links**10.20.15.2.1 Limitations**

Links shall meet the requirements of Sec 10.20.8.2.2.

The web of a link shall be single thickness. Doubler-plate reinforcement and web penetrations are not permitted.

10.20.15.2.2 Shear strength

Except as limited below, the link design shear strength, $\phi_v V_n$ and the allowable shear strength, V_n/Ω_v , according to the limit state of shear yielding shall be determined as follows:

V_n = nominal shear strength of the link, equal to the lesser of V_p or $2M_p/e$, N

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

Where,

$$M_p = F_y Z, \text{ N-mm}$$

$$V_p = 0.6 F_y A_w, \text{ N}$$

$$e = \text{link length, mm}$$

$$A_w = (d - 2t_f)t_w$$

The effect of axial force on the link available shear strength need not be considered if

$$P_u \leq 0.15P_y \quad (\text{LRFD}) \quad \text{or} \quad P_a \leq (0.15/1.5)P_y \quad (\text{ASD}),$$

as appropriate.

Where,

P_u = required axial strength using LRFD load combinations, N

P_a = required axial strength using ASD load combinations, N

P_y = nominal axial yield strength = $F_y A_g$, N

If $P_u > 0.15P_y$ (LRFD) or $P_a > (0.15/1.5)P_y$ (ASD), as appropriate, the following additional requirements shall be met:

- (1) The available shear strength of the link shall be the lesser of

$$\phi_v V_{pa} \text{ and } 2\phi_v M_{pa}/e \quad (\text{LRFD})$$

Or,

$$V_{pa}/\Omega_v \text{ and } 2(M_{pa}/e)/\Omega_v \quad (\text{ASD}), \text{ as appropriate,}$$

Where,

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

$$V_{pa} = V_p \sqrt{(1 - (P_r/P_c)^2)} \quad (6.10.303)$$

$$M_{pa} = 1.18M_p[1 - (P_r/P_c)] \quad (6.10.304)$$

$$P_r = P_u \quad (\text{LRFD}) \text{ or } P_a \quad (\text{ASD}), \text{ as appropriate}$$

$$P_c = P_y \quad (\text{LRFD}) \text{ or } P_y/1.5 \quad (\text{ASD}), \text{ as appropriate}$$

(2) The length of the link shall not exceed:

$$(a) \quad [1.15 - 0.5\rho'(A_w/A_g)]1.6M_p/V_p \text{ when } \rho'(A_w/A_g) \geq 0.3$$

(6.10.305)

Nor,

$$(b) \quad 1.6M_p/V_p \text{ when } \rho'(A_w/A_g) < 0.3$$

(6.10.306)

Where,

$$A_w = (d - 2t_f)t_w$$

$$\rho' = P_r/V_r$$

And where,

$$V_r = V_u \text{ (LRFD) or } V_r = V_a \text{ (ASD), as appropriate}$$

$$V_u = \text{required shear strength based on LRFD load combinations.}$$

$$V_a = \text{required shear strength based on ASD load combinations.}$$

10.20.15.2.3 Link rotation angle

The link rotation angle is the inelastic angle between the link and the beam outside of the link when the total story drift is equal to the design story drift, Δ . The link rotation angle shall not exceed the following values:

- (a) 0.08 radians for links of length $1.6M_p/V_p$ or less.
- (b) 0.02 radians for links of length $2.6M_p/V_p$ or greater.
- (c) The value determined by linear interpolation between the above values for links of length between $1.6M_p/V_p$ and $2.6M_p/V_p$.

10.20.15.3 Link stiffeners

Full-depth web stiffeners shall be provided on both sides of the link web at the diagonal brace ends of the link. These stiffeners shall have a combined width not less than $(b_f - 2t_w)$ and a thickness not less than $0.75t_w$ or 10 mm, whichever is larger, where b_f and t_w are the link flange width and link web thickness, respectively.

Links shall be provided with intermediate web stiffeners as follows:

- (a) Links of lengths $1.6M_p/V_p$ or less shall be provided with intermediate web stiffeners spaced at intervals not exceeding $(30t_w - d/5)$ for a link rotation angle of 0.08 radian or $(52t_w - d/5)$ for link rotation angles of 0.02 radian or less. Linear interpolation shall be used for values between 0.08 and 0.02 radian.
- (b) Links of length greater than $2.6M_p/V_p$ and less than $5M_p/V_p$ shall be provided with intermediate web stiffeners placed at a distance of 1.5 times b_f from each end of the link.
- (c) Links of length between $1.6M_p/V_p$ and $2.6M_p/V_p$ shall be provided with intermediate web stiffeners meeting the requirements of (a) and (b) above.
- (d) Intermediate web stiffeners are not required in links of lengths greater than $5M_p/V_p$.
- (e) Intermediate web stiffeners shall be full depth. For links that are less than 635 mm in depth, stiffeners are required on only one side of the link web. The thickness of one-sided stiffeners shall not be less than t_w or 10 mm, whichever is larger, and the width shall be not less than $(b_f/2)t_w$. For links that are 635 mm in depth or greater, similar intermediate stiffeners are required on both sides of the web.

The required strength of fillet welds connecting a link stiffener to the link web is $A_{st}F_y$ (LRFD) or $A_{st}F_y/1.5$ (ASD), as appropriate, where A_{st} is the area of the stiffener. The required strength of fillet welds connecting the stiffener to the link flanges is $A_{st}F_y/4$ (LRFD) or $A_{st}F_y/4(1.5)$ (ASD).

10.20.15.4 Link-to-column connections

Link-to-column connections must be capable of sustaining the maximum link rotation angle based on the length of the link, as specified in Sec 10.20.15.2.3. The strength of the connection measured at the column face shall equal at least the nominal shear strength of

the link, V_n , as specified in Sec 10.20.15.2.2 at the maximum link rotation angle. Link-to-column connections shall satisfy the above requirements by one of the following:

- (a) Use a connection prequalified for EBF in accordance with Appendix N.
- (b) Provide qualifying cyclic test results in accordance with Appendix Q. Results of at least two cyclic connection tests shall be provided and are permitted to be based on one of the following:
 - (i) Tests reported in research literature or documented tests performed for other projects that are representative of project conditions, within the limits specified in Appendix Q.
 - (ii) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes, within the limits specified in Appendix Q.

Exception:

Where reinforcement at the beam-to-column connection at the link end precludes yielding of the beam over the reinforced length, the link is permitted to be the beam segment from the end of the reinforcement to the brace connection. Where such links are used and the link length does not exceed $1.6M_p/V_p$, cyclic testing of the reinforced connection is not required if the available strength of the reinforced section and the connection equals or exceeds the required strength calculated based upon the strain-hardened link as described in Sec 10.20.15.6. Full depth stiffeners as required in Sec 10.20.15.3 shall be placed at the link-to-reinforcement interface.

10.20.15.5 Lateral bracing of link

Lateral bracing shall be provided at both the top and bottom link flanges at the ends of the link. The required strength of each lateral brace at the ends of the link shall be $P_b = 0.06M_r/h_o$, where, h_o is the distance between flange centroids, in mm.

For design according to Specification Sec 10.2.3.3 (LRFD)

$$M_r = M_{u,exp} = R_y Z F_y$$

For design according to Specification Sec 10.2.3.4 (ASD)

$$M_r = M_{u,exp}/1.5$$

The required brace stiffness shall meet the provisions of Eq. 6.10.292 of Sec 10.19, where M_r is defined above, $C_d = 1$, and L_b is the link length.

10.20.15.6 Diagonal brace and beam outside of link

10.20.15.6.1 Diagonal brace

The required combined axial and flexural strength of the diagonal brace shall be determined based on load combinations stipulated by the Code. For load combinations including seismic effects, a load Q_1 shall be substituted for the term E , where Q_1 is defined as the axial forces and moments generated by at least 1.25 times the expected nominal shear strength of the link $R_y V_n$, where V_n is as defined in Sec 10.20.15.2.2. The available strength of the diagonal brace shall comply with Specification Sec 10.10. Brace members shall meet the requirements of Sec 10.20.8.2.1.

10.20.15.6.2 Beam outside link

The required combined axial and flexural strength of the beam outside of the link shall be determined based on load combinations stipulated by the Code. For load combinations including seismic effects, a load Q_1 shall be substituted for the term E where Q_1 is defined as the forces generated by at least 1.1 times the expected nominal shear strength of the link, $R_y V_n$, where V_n is as defined in Sec 10.20.15.2.2. The available strength of the beam outside of the link shall be determined by the Specification, multiplied by R_y .

At the connection between the diagonal brace and the beam at the link end of the brace, the intersection of the brace and beam centerlines shall be at the end of the link or in the link.

10.20.15.6.3 Bracing connections

The required strength of the diagonal brace connections, at both ends of the brace, shall be at least equal to the required strength of the diagonal brace, as defined in Sec 10.20.15.6.1. The diagonal brace connections shall also satisfy the requirements of Sec 10.20.13.3.3.

No part of the diagonal brace connection at the link end of the brace shall extend over the link length. If the brace is designed to resist a portion of the link end moment, then the diagonal brace connection at the link end of the brace shall be designed as a fully-restrained moment connection.

10.20.15.7 Beam-to-column connections

If the EBF system factors in the Code require moment resisting connections away from the link, then the beam-to-column connections away from the link shall meet the requirements

for beam-to-column connections for OMF specified in Sections 10.20.11.2 and 10.20.11.5.

If EBF system factors in the Code do not require moment resisting connections away from the link, then the beam-to-column connections away from the link are permitted to be designed as pinned in the plane of the web.

10.20.15.8 Required strength of columns

In addition to the requirements in Sec 10.20.8.3, the required strength of columns shall be determined from load combinations as stipulated by the Code, except that the seismic load E shall be the forces generated by 1.1 times the expected nominal shear strength of all links above the level under consideration. The expected nominal shear strength of a link is $R_y V_n$, where V_n is as defined in Sec 10.20.15.2.2. Column members shall meet the requirements of Sec 10.20.8.2.2.

10.20.15.9 Protected zone

Links in EBFs are a protected zone, and shall satisfy the requirements of Sec 10.20.7.4. Welding on links is permitted for attachment of link stiffeners, as required in Sec 10.20.15.3.

10.20.15.10 Demand critical welds

Complete-joint-penetration groove welds attaching the link flanges and the link web to the column are demand critical welds, and shall satisfy the requirements of Sec 10.20.7.3.2.

10.20.16 Buckling-Restrained Braced Frames (BRBF)

10.20.16.1 Scope

Buckling-restrained braced frames (BRBF) are expected to withstand significant inelastic deformations when subjected to the forces resulting from the motions of the design earthquake. BRBF shall meet the requirements in this Section. Where the Code does not contain design coefficients for BRBF, provisions of Appendix P shall apply.

10.20.16.2 Bracing members

Bracing members shall be composed of a structural steel core and a system that restrains the steel core from buckling.

10.20.16.2.1 Steel core

The steel core shall be designed to resist the entire axial force in the brace.

The brace design axial strength, $\phi P_{y_{sc}}$ (LRFD), and the brace allowable axial strength, $P_{y_{sc}}/\Omega$ (ASD), in tension and compression, according to the limit state of yielding, shall be determined as follows:

$$P_{y_{sc}} = F_{y_{sc}} = A_{sc} \quad (\text{Sec 10.20.16.1})$$

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

Where,

$F_{y_{sc}}$ = specified minimum yield stress of the steel core, or actual yield stress of the steel core as determined from a coupon test, MPa

A = net area of steel core, mm²

Plates used in the steel core that are 50 mm thick or greater shall satisfy the minimum notch toughness requirements of Sec 10.20.6.3.

Splices in the steel core are not permitted.

10.20.16.2.2 Buckling-restraining system

The buckling-restraining system shall consist of the casing for the steel core. In stability calculations, beams, columns, and gussets connecting the core shall be considered parts of this system.

The buckling-restraining system shall limit local and overall buckling of the steel core for deformations corresponding to 2.0 times the design story drift. The buckling-restraining system shall not be permitted to buckle within deformations corresponding to 2.0 times the design story drift.

10.20.16.2.3 Testing

The design of braces shall be based upon results from qualifying cyclic tests in accordance with the procedures and acceptance criteria of Appendix R. Qualifying test results shall consist of at least two successful cyclic tests: one is required to be a test of a brace sub-assembly that includes brace connection rotational demands complying with Appendix R, Sec R.4 and the other shall be either a uniaxial or a sub-assembly test complying with

Appendix R, Sec R.5. Both test types are permitted to be based upon one of the following:

- (a) Tests reported in research or documented tests performed for other projects.
- (b) Tests that are conducted specifically for the project.

Interpolation or extrapolation of test results for different member sizes shall be justified by rational analysis that demonstrates stress distributions and magnitudes of internal strains consistent with or less severe than the tested assemblies and that considers the adverse effects of variations in material properties. Extrapolation of test results shall be based upon similar combinations of steel core and buckling-restraining system sizes. Tests shall be permitted to qualify a design when the provisions of Appendix R are met.

10.20.16.2.4 Adjusted brace strength

Where required by these Provisions, bracing connections and adjoining members shall be designed to resist forces calculated based on the adjusted brace strength.

The adjusted brace strength in compression shall be $\beta\omega R_y P_{y_{sc}}$. The adjusted brace strength in tension shall be $\omega R_y P_{y_{sc}}$.

Exception:

The factor R_y need not be applied if $P_{y_{sc}}$ is established using yield stress determined from a coupon test. The compression strength adjustment factor, β , shall be calculated as the ratio of the maximum compression force to the maximum tension force of the test specimen measured from the qualification tests specified in Appendix R, Sec R.6.3 for the range of deformations corresponding to 2.0 times the design story drift. The larger value of β from the two required brace qualification tests shall be used. In no case shall β be taken as less than 1.0. The strain hardening adjustment factor, ω , shall be calculated as the ratio of the maximum tension force measured from the qualification tests specified in Appendix R, Sec R.6.3 (for the range of deformations corresponding to 2.0 times the design story drift) to $F_{y_{sc}}$ of the test specimen. The larger value of ω from the two required qualification tests shall be used. Where the tested steel core material does not match that of the prototype, ω shall be based on coupon testing of the prototype material.

10.20.16.3 Bracing connections

10.20.16.3.1 Required strength

The required strength of bracing connections in tension and compression (including beam-to-column connections if part of the bracing system) shall be 1.1 times the adjusted brace strength in compression (LRFD) or $(1.1/1.5)$ times the adjusted brace strength in compression (ASD).

10.20.16.3.2 Gusset plates

The design of connections shall include considerations of local and overall buckling. Bracing consistent with that used in the tests upon which the design is based is required.

10.20.16.4 Special requirements related to bracing configuration

V-type and inverted-V-type braced frames shall meet the following requirements:

- (1) The required strength of beams intersected by braces, their connections, and supporting members shall be determined based on the load combinations of the Code assuming that the braces provide no support for dead and live loads. For load combinations that include earthquake effects, the vertical and horizontal earthquake effect, E , on the beam shall be determined from the adjusted brace strengths in tension and compression.
- (2) Beams shall be continuous between columns. Both flanges of beams shall be laterally braced. Lateral braces shall meet the provisions of Equations 6.10.291 and 6.10.292 of Sec 10.19, where, $M_r = M_u = R_y Z F_y$ (LRFD) or $M_r = M_a = R_y Z F_y / 1.5$ (ASD), as appropriate, of the beam and $C_d = 1.0$. As a minimum, one set of lateral braces is required at the point of intersection of the V-type (or inverted V-type) bracing, unless the beam has sufficient out-of-plane strength and stiffness to ensure stability between adjacent brace points.

For purposes of brace design and testing, the calculated maximum deformation of braces shall be increased by including the effect of the vertical deflection of the beam under the loading defined in Sec 10.20.16.4(1).

K-type braced frames are not permitted for BRBF.

10.20.16.5 Beams and columns

Beams and columns in BRBF shall meet the following requirements.

10.20.16.5.1 Width-thickness limitations

Beam and column members shall meet the requirements of Sec 10.20.8.2.2.

10.20.16.5.2 Required strength

The required strength of beams and columns in BRBF shall be determined from load combinations as stipulated in the Code. For load combinations that include earthquake effects, the earthquake effect, E , shall be determined from the adjusted brace strengths in tension and compression.

The required strength of beams and columns need not exceed the maximum force that can be developed by the system.

10.20.16.5.3 Splices

In addition to meeting the requirements in Sec 10.20.8.4, column splices in BRBF shall be designed to develop 50 percent of the lesser available flexural strength of the connected members, determined based on the limit state of yielding. The required shear strength shall be $\sum M_{pc}/H$ (LRFD) or $\sum M_{pc}/1.5H$ (ASD), as appropriate, where $\sum M_{pc}$ is the sum of the nominal plastic flexural strengths of the columns above and below the splice.

10.20.16.6 Protected zone

The protected zone shall include the steel core of bracing members and elements that connect the steel core to beams and columns, and shall satisfy the requirements of Sec 10.20.7.4.

10.20.17 Special Plate Shear Walls (SPSW)

10.20.17.1 Scope

Special plate shear walls (SPSW) are expected to withstand significant inelastic deformations in the webs when subjected to the forces resulting from the motions of the design earthquake. The horizontal boundary elements (HBEs) and vertical boundary elements (VBEs) adjacent to the webs shall be designed to remain essentially elastic under the maximum forces that can be generated by the fully yielded webs, except that plastic hinging at the ends of HBEs is permitted. SPSW shall meet the requirements of this Section. Where the Code does not contain design coefficients for SPSW, the provisions of Appendix P shall apply.

10.20.17.2 Webs

10.20.17.2.1 Shear strength

The panel design shear strength, ϕV_n (LRFD), and the allowable shear strength, V_n/Ω (ASD), according to the limit state of shear yielding, shall be determined as follows:

$$V_n = 0.42F_y t_w L_{cf} \sin 2\alpha \quad (6.10.307)$$

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

Where,

t_w = thickness of the web, mm.

L_{cf} = clear distance between VBE flanges, mm

α = angle of web yielding in radians, as measured relative to the vertical, and it is given by:

$$\tan^4 \alpha = \frac{1 + \frac{t_w L}{2A_c}}{1 + t_w h \left(\frac{1}{A_b} + \frac{h^3}{360I_c L} \right)} \quad (6.10.308)$$

h = distance between HBE centerlines, mm

A_b = cross-sectional area of a HBE, mm²

A_c = cross-sectional area of a VBE, mm²

I_c = moment of inertia of a VBE taken perpendicular to the direction of the web plate line, mm⁴

L = distance between VBE centerlines, mm

10.20.17.2.2 Panel aspect ratio

The ratio of panel length to height, L/h , shall be limited to $0.8 < L/h \leq 2.5$.

10.20.17.2.3 Openings in webs

Openings in webs shall be bounded on all sides by HBE and VBE extending the full width and height of the panel, respectively, unless otherwise justified by testing and analysis.

10.20.17.3 Connections of webs to boundary elements

The required strength of web connections to the surrounding HBE and VBE shall equal the expected yield strength, in tension, of the web calculated at an angle α , defined by Eq. 6.10.308.

10.20.17.4 Horizontal and vertical boundary elements

10.20.17.4.1 Required strength

In addition to the requirements of Sec 10.20.8.3, the required strength of VBE shall be based upon the forces corresponding to the expected yield strength, in tension, of the web calculated at an angle α .

The required strength of HBE shall be the greater of the forces corresponding to the expected yield strength, in tension, of the web calculated at an angle α or that determined from the load combinations in the Code assuming the web provides no support for gravity loads.

The beam-column moment ratio provisions in Sec 10.20.9.6 shall be met for all HBE/VBE intersections without consideration of the effects of the webs.

10.20.17.4.2 HBE-to-VBE connections

HBE-to-VBE connections shall satisfy the requirements of Sec 10.20.11.2. The required shear strength, V_u , of a HBE-to-VBE connection shall be determined in accordance with the provisions of Sec 10.20.11.2, except that the required shear strength shall not be less than the shear corresponding to moments at each end equal to $1.1R_yM_p$ (LRFD) or $(1.1/1.5)R_yM_p$ (ASD), as appropriate, together with the shear resulting from the expected yield strength in tension of the webs yielding at an angle α .

10.20.17.4.3 Width-thickness limitations

HBE and VBE members shall meet the requirements of Sec 10.20.8.2.2.

10.20.17.4.4 Lateral bracing

HBE shall be laterally braced at all intersections with VBE and at a spacing not to exceed $0.086r_y E/F_y$. Both flanges of HBE shall be braced either directly or indirectly. The required strength of lateral bracing shall be at least 2 percent of the HBE flange nominal strength, $F_y b_f t_f$. The required stiffness of all lateral bracing shall be determined in accordance with Eq. 6.10.292 of Sec 10.19. In these Equations, M_r shall be computed as $R_y Z F_y$ (LRFD) or M_r shall be computed as $R_y Z F_y / 1.5$ (ASD), as appropriate, and $C_d = 1.0$.

10.20.17.4.5 VBE splices

VBE splices shall comply with the requirements of Sec 10.20.8.4.

10.20.17.4.6 Panel zones

The VBE panel zone next to the top and base HBE of the SPSW shall comply with the requirements in Sec 10.20.9.3.

10.20.17.4.7 Stiffness of vertical boundary elements

The VBE shall have moments of inertia about an axis taken perpendicular to the plane of the web, I_c not less than $0.00307 t_w h^4 / L$.

10.20.18 Quality Assurance Plan**10.20.18.1 Scope**

When required by the Code or the Engineer, a quality assurance plan shall be provided. The quality assurance plan shall include the requirements of Appendix O.

10.21 List of Related Appendices

Appendix N	Prequalification of Beam-Column and Link-to-Column Connections
Appendix O	Quality Assurance Plan
Appendix P	Seismic Design Coefficients and Approximate Period Parameters
Appendix Q	Qualifying Cyclic Tests of Beam-to-Column and Link-to-Column Connections
Appendix R	Qualifying Cyclic Tests of Buckling-restrained Braces
Appendix S	Welding Provisions
Appendix T	Weld Metal/Welding Procedure Specification Notch Toughness Verification Test

PART VI**Chapter 11****TIMBERS STRUCTURES****11.1 scope**

11.1.1 This Section relates to the use of structural timber in structures or elements of structures connected together by fasteners/fastening techniques.

11.1.2 This shall not be interpreted to prevent the use of material or methods of design or construction not specifically mentioned herein; and the methods of design may be based on analytical and engineering principles, or reliable test data, or both, that demonstrate the safety and serviceability of the resulting structure. Nor is the classification of timber into strength groups to be interpreted as preventing the use of design data desired for a particular timber or grade of timber on the basis of reliable tests.

11.2 Terminology

11.2.1 This Section provides an alphabetical list of the terms used in this Chapter of the Code. In case of any conflict or contradiction between a definition given in this Section and that in Part 1, the meaning provided in this Section shall govern for interpretation of the provisions of this Chapter.

11.2.2 Structural Purpose Definitions

BUILT-UP-LAMINATED BEAM	A beam made by joining layers of timber together with mechanical fastenings, so that the grain of all layers is essentially parallel.
DURATION OF LOAD	Period during which a member or a complete structure is stressed as a consequence of the loads applied.
EDGE DISTANCE	The distance measured perpendicular to grain from the centre of the connector to the edge of the member.
END DISTANCE	The distance measured parallel to grain of the member from the centre of the connector to the closest end of timber.
FINGER JOINT	Joint produced by connecting timber members end-to-end by cutting profiles (tapered projections) in the form of V-shaped grooves to the ends of timber planks or scantlings to be joined, gluing the interfaces and then mating the two ends together under pressure.