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# Specification for Structural Steel Buildings

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**PUBLIC REVIEW DRAFT DATED March 1, 2009**

Supersedes the  
*Specification for Structural Steel Buildings*  
dated March 9, 2005  
and all previous versions of this specification.

Approved by the AISC Committee on Specifications and issued by the  
AISC Board of Directors



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## PREFACE (to be revised)

(This Preface is not part of ANSI/AISC 360-??, *Specification for Structural Steel Buildings*, but is included for informational purposes only.)

This Specification has been based upon past successful usage, advances in the state of knowledge, and changes in design practice. The 2005 American Institute of Steel Construction's *Specification for Structural Steel Buildings* for the first time provides an integrated treatment of Allowable Stress Design (ASD) and Load and Resistance Factor Design (LRFD), and thus combines and replaces earlier Specifications that treated the two design methods separately. As indicated in Chapter B of the Specification, designs can be made according to either ASD or LRFD provisions.

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

This Specification is the result of the consensus deliberations of a committee of structural engineers with wide experience and high professional standing, representing a wide geographical distribution throughout the United States. The committee includes approximately equal numbers of engineers in private practice and code agencies, engineers involved in research and teaching, and engineers employed by steel fabricating and producing companies. The contributions and assistance of more than 50 additional professional volunteers working in ten task committees are also hereby acknowledged.

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

The reader is cautioned that professional judgment must be exercised when data or recommendations in the Specification are applied, as described more fully in the disclaimer notice preceding this Preface.

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# Symbols

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

Symbol	Definition	Section
$A$	Cross-sectional area of column, in. <sup>2</sup> (mm <sup>2</sup> ).....	J10.6
$A$	Total cross-sectional area of member, in. <sup>2</sup> (mm <sup>2</sup> ).....	E7.2
$A_l$	Loaded area of concrete, in. <sup>2</sup> (mm <sup>2</sup> ).....	I5.1
$A_{BM}$	Cross-sectional area of the base metal, in. <sup>2</sup> (mm <sup>2</sup> ) .....	J2.4
$A_b$	Nominal unthreaded body area of bolt or threaded part, in. <sup>2</sup> (mm <sup>2</sup> ) .....	J3.6
$A_{bi}$	Cross-sectional area of the overlapping branch, in. <sup>2</sup> (mm <sup>2</sup> ) .....	K2.3
$A_{bj}$	Cross-sectional area of the overlapped branch, in. <sup>2</sup> (mm <sup>2</sup> ) .....	K2.3
$A_c$	Area of concrete, in. <sup>2</sup> (mm <sup>2</sup> ) .....	I2.1
$A_c$	Area of concrete slab within effective width, in. <sup>2</sup> (mm <sup>2</sup> ) .....	I3.2
$A_D$	Area of upset rod based on the major thread diameter, in. <sup>2</sup> (mm <sup>2</sup> ) .....	Table J3.2
$A_e$	Effective net area, in. <sup>2</sup> (mm <sup>2</sup> ).....	D2
$A_e$	Summation of the effective areas of the cross section based on the reduced effective width, $b_e$ , in. <sup>2</sup> (mm <sup>2</sup> ).....	E7.2
$A_{fc}$	Area of compression flange .....	G3.1
$A_{fg}$	Gross area of tension flange, in. <sup>2</sup> (mm <sup>2</sup> ).....	F13.1
$A_{fn}$	Net area of tension flange, in. <sup>2</sup> (mm <sup>2</sup> ).....	F13.1
$A_{ft}$	Area of tension flange, in. <sup>2</sup> (mm <sup>2</sup> ) .....	G3.1
$A_g$	Gross area of member, in. <sup>2</sup> (mm <sup>2</sup> ).....	B4.3
$A_g$	Gross area of section based on design wall thickness, in. <sup>2</sup> (mm <sup>2</sup> ) .....	G6
$A_g$	Gross area of composite member, in. <sup>2</sup> (mm <sup>2</sup> ) .....	I2.1
$A_g$	Gross section of connected material, in. <sup>2</sup> (mm <sup>2</sup> ).....	J3.6
$A_{gv}$	Gross area subject to shear, in. <sup>2</sup> (mm <sup>2</sup> ).....	J4.2
$A_n$	Net area of member, in. <sup>2</sup> (mm <sup>2</sup> ) .....	B4.3
$A_n$	Effective net area of connected material, in. <sup>2</sup> (mm <sup>2</sup> ) .....	J3.6
$A_{nt}$	Net area subject to tension, in. <sup>2</sup> (mm <sup>2</sup> ) .....	J4.3
$A_{nv}$	Net area subject to shear, in. <sup>2</sup> (mm <sup>2</sup> ).....	J4.2
$A_{pb}$	Projected area in bearing, in. <sup>2</sup> (mm <sup>2</sup> ).....	J7
$A_s$	Area of steel cross section, in. <sup>2</sup> (mm <sup>2</sup> ) .....	I2.1
$A_s$	Total bolt area included in all shear planes. With threads in the shear plane, $A_s$ is taken as 0.80 times the nominal area of the bolts, in. <sup>2</sup> (mm <sup>2</sup> ) .....	J3.6
$A_{sc}$	Cross-sectional area of stud connector, in. <sup>2</sup> (mm <sup>2</sup> ) .....	I7.2
$A_{sf}$	Area on the shear failure path, in. <sup>2</sup> (mm <sup>2</sup> ).....	D5.1
$A_{sr}$	Area of continuous reinforcing bars, in. <sup>2</sup> (mm <sup>2</sup> ) .....	I2.1
$A_{sr}$	Area of adequately developed longitudinal reinforcing	



946	steel within the effective width of the concrete slab,	
947	in. <sup>2</sup> (mm <sup>2</sup> ) .....	I3.2
948	$A_t$ Net area in tension, in. <sup>2</sup> (mm <sup>2</sup> ).....	App. 3.4
949	$A_w$ Area of web, the overall depth times the web	
950	thickness, $dt_w$ , in. <sup>2</sup> (mm <sup>2</sup> ).....	G2.1
951	$A_{we}$ Effective area of weld, in. <sup>2</sup> (mm <sup>2</sup> ).....	J2.4
952	$A_{wei}$ Effective area of weld throat of any $i$ th weld element,	
953	in. <sup>2</sup> (mm <sup>2</sup> ) .....	J2.4
954	$A_1$ Area of steel concentrically bearing on a concrete	
955	support, in. <sup>2</sup> (mm <sup>2</sup> ).....	J8
956	$A_2$ Maximum area of the portion of the supporting surface	
957	that is geometrically similar to and concentric with	
958	the loaded area, in. <sup>2</sup> (mm <sup>2</sup> ).....	J8
959	$B$ Overall width of rectangular HSS main member,	
960	measured 90 degrees to the plane of the connection,	
961	in. (mm) .....	Table D3.1, K3.1
962	$B$ Factor for lateral-torsional buckling in tees and double	
963	angles .....	F9.2
964	$B_b$ Overall width of rectangular HSS branch member,	
965	measured 90 degrees to the plane of the connection,	
966	in. (mm) .....	K3.1
967	$B_{bi}$ Overall width of the overlapping branch .....	K2.3
968	$B_{bj}$ Overall width of the overlapped branch .....	K2.3
969	$B_p$ Width of plate, measure 90 degrees to the plane of the	
970	connection, in. (mm) .....	K1.1
971	$B_1, B_2$ Factors used in determining $M_u$ for combined bending	
972	and axial forces when first-order analysis is em-	
973	ployed .....	App.8.2
974	$C$ HSS torsional constant.....	H3.1
975	$C_1$ Coefficient for calculation of effective rigidity of an	
976	encased composite compression member .....	I2.1b
977	$C_3$ Coefficient for calculation of effective rigidity of a	
978	filled composite compression member.....	I2.2b
979	$C_b$ Lateral-torsional buckling modification factor for	
980	nonuniform moment diagrams when both ends of the	
981	unsupported segment are braced.....	F1
982	$C_d$ Coefficient relating relative brace stiffness and	
983	curvature .....	App. 6.3.1
984	$C_f$ Constant based on stress category, given in Table A-	
985	3.1 .....	App. 3.3
986	$C_{in}$ Constant used to calculate direct bond strength	
987	dependent on the location of a filled composite	
988	member with respect to the point of load transfer .....	I5.3c
989	$C_m$ Coefficient assuming no lateral translation of the	
990	frame .....	App. 8.2.1
991	$C_p$ Ponding flexibility coefficient for primary member in	
992	a flat roof .....	App. 2.1
993	$C_r$ Coefficient for web sidesway buckling .....	J10.4
994	$C_s$ Ponding flexibility coefficient for secondary member	
995	in a flat roof .....	App. 2.1

996	$C_v$	Web shear coefficient .....	G2.1
997	$C_w$	Warping constant, in. <sup>6</sup> (mm <sup>6</sup> ) .....	E4
998	$D$	Nominal dead load .....	App.4.1.4
999	$D$	Chord diameter, in. (mm).....	Table B4.1
1000	$D$	Outside diameter, in. (mm) .....	E7.2
1001	$D$	Outside diameter of round HSS main member, in.	
1002		(mm) .....	K2.1
1003	$D_b$	Outside diameter of round HSS branch member, in.	
1004		(mm) .....	K2.1
1005	$D_u$	In slip-critical connections, a multiplier that reflects	
1006		the ratio of the mean installed bolt pretension to the	
1007		specified minimum bolt pretension .....	J3.8
1008	$E$	Modulus of elasticity of steel = 29,000 ksi (200 000	
1009		MPa) .....	Table B4.1
1010	$E_c$	Modulus of elasticity of concrete = $w_c^{1.5} \sqrt{f'_c}$ , ksi	
1011		( $0.043 w_c^{1.5} \sqrt{f'_c}$ , MPa) .....	I2.1
1012	$E_{cm}$	Modulus of elasticity of concrete at elevated	
1013		temperature, ksi (MPa) .....	App. 4.2.3
1014	$E_m$	Modulus of elasticity of steel at elevated temperature,	
1015		ksi (MPa) .....	App. 4.2.3
1016	$E_s$	Modulus of elasticity of steel = 29,000 ksi (200 000	
1017		MPa) .....	I2.1
1018	$EI_{eff}$	Effective stiffness of composite section, kip-in. <sup>2</sup> (N-	
1019		mm <sup>2</sup> ) .....	I2.1
1020	$F_c$	Available stress in chord, ksi (MPa) .....	K1
1021	$F_{cr}$	Flexural buckling stress, ksi (MPa) .....	E3
1022	$F_{ca}$	Available axial stress at the point of consideration, ksi	
1023		(MPa) .....	H2
1024	$F_{cbw}$	Available flexural stress at the point of consideration	
1025		about the major axis, ksi (MPa) .....	H2
1026	$F_{cbz}$	Available flexural stress at the point of consideration	
1027		about the minor axis, ksi (MPa) .....	H2
1028	$F_{cr}$	Buckling stress for the section as determined by	
1029		analysis, ksi (MPa).....	F12.2
1030	$F_{cry}$	Critical stress about the minor axis, ksi (MPa) .....	E4
1031	$F_{crz}$	Critical torsional buckling stress, ksi (MPa).....	E4
1032	$F_e$	Elastic critical buckling stress, ksi (MPa) .....	E3
1033	$F_{ex}$	Elastic flexural buckling stress about the major axis,	
1034		ksi (MPa) .....	E4
1035	$F_{EXX}$	Electrode classification number, ksi (MPa) .....	J2.4
1036	$F_{ey}$	Elastic flexural buckling stress about the minor axis,	
1037		ksi (MPa) .....	E4
1038	$F_{ez}$	Elastic torsional buckling stress, ksi (MPa).....	E4
1039	$F_{in}$	Nominal bond stress, 0.06 ksi (0.40MPa).....	F5.2c
1040	$F_L$	Calculated stress used in the determination of nominal	
1041		flexural strength, ksi (MPa) .....	Table B4.1
1042	$F_n$	Nominal torsional strength.....	H3.3
1043	$F_n$	Nominal tensile stress $F_{nt}$ , or shear stress, $F_{nv}$ , from	
1044		Table J3.2, ksi (MPa).....	J3.6

1045	$F_{nBM}$	Nominal stress of the base metal, ksi (MPa).....	J2.4
1046	$F_{nt}$	Nominal tensile stress from Table J3.2, ksi (MPa).....	J3.7
1047	$F'_{nt}$	Nominal tensile stress modified to include the effects	
1048		of shear stress, ksi (MPa).....	J3.7
1049	$F_{nv}$	Nominal shear stress from Table J3.2, ksi (MPa).....	J3.7
1050	$F_{nw}$	Nominal stress of the weld metal, ksi (MPa).....	J2.4
1051	$F_{nwi}$	Nominal stress in <i>i</i> th weld element, ksi (MPa).....	J2.4
1052	$F_{nwi,x}$	x component of stress $F_{nwi}$ , ksi (MPa).....	J2.4
1053	$F_{nwi,y}$	y component of stress $F_{nwi}$ , ksi (MPa).....	J2.4
1054	$F_{SR}$	Service stress range, ksi (MPa) .....	App. 3.3
1055	$F_{TH}$	Threshold fatigue stress range, maximum stress range	
1056		for indefinite design life from Table A-3.1, ksi	
1057		(MPa) .....	App. 3.1
1058	$F_u$	Specified minimum tensile strength of the type of steel	
1059		being used, ksi (MPa).....	D2
1060	$F_u$	Specified minimum tensile strength of a stud shear	
1061		connector, ksi (MPa).....	I7.2
1062	$F_u$	Specified minimum tensile strength of the connected	
1063		material, ksi (MPa) .....	J3.10
1064	$F_u$	Specified minimum tensile strength of HSS material,	
1065		ksi (MPa) .....	K1.1
1066	$F_{ub}$	Nominal tensile strength of bolt or threaded material,	
1067		ksi (MPa).....	J3.6
1068	$F_{um}$	Specified minimum tensile strength of the type of steel	
1069		being used at elevated temperature, ksi (MPa).....	App. 4.2
1070	$F_w$	Nominal strength of the weld metal per unit area	
1071		(Chapter J) with no increase in strength due to direc-	
1072		tionality of load, ksi (MPa).....	K4
1073	$F_y$	Specified minimum yield stress of the type of steel	
1074		being used, ksi (MPa). As used in this Specification,	
1075		“yield stress” denotes either the specified minimum	
1076		yield point (for those steels that have a yield point) or	
1077		specified yield strength (for those steels that do not	
1078		have a yield point).....	Table B4.1
1079	$F_y$	Specified minimum yield stress of the compression	
1080		flange, ksi (MPa) .....	App. 1.3
1081	$F_y$	Specified minimum yield stress of the connected	
1082		material, ksi (MPa) .....	J3.6
1083	$F_y$	Specified minimum yield stress of the column web, ksi	
1084		(MPa) .....	J10.6
1085	$F_y$	Specified minimum yield stress of HSS member	
1086		material, ksi (MPa) .....	K1.1
1087	$F_y$	Specified minimum yield stress of HSS main member	
1088		material, ksi (MPa) .....	K2.1
1089	$F_{yb}$	Specified minimum yield stress of HSS branch	
1090		member material, ksi (MPa) .....	K2.1
1091	$F_{ybi}$	Specified minimum yield stress of the overlapping	
1092		branch material, ksi (MPa) .....	K2.3
1093	$F_{ybj}$	Specified minimum yield stress of the overlapped	
1094		branch material, ksi (MPa) .....	K2.3

1095	$F_{yf}$	Specified minimum yield stress of the flange, ksi	
1096		(MPa) .....	J10.1
1097	$F_{ym}$	Specified minimum yield stress of the type of steel	
1098		being used at elevated temperature, ksi (MPa) .....	App. 4.2
1099	$F_{yp}$	Specified minimum yield stress of plate, ksi (MPa) .....	K1.1
1100	$F_{ysr}$	Specified minimum yield stress of reinforcing bars,	
1101		ksi (MPa) .....	I2.1
1102	$F_{yst}$	Specified minimum yield stress of the stiffener	
1103		material, ksi (MPa).....	G3.3
1104	$F_{yw}$	Specified minimum yield stress of the web, ksi (MPa) .....	J10.2
1105	$G$	Shear modulus of elasticity of steel = 11,200 ksi	
1106		(77 200 MPa) .....	E4
1107	$\Sigma H$	Story shear produced by the lateral forces used to	
1108		compute $\Delta_H$ , kips (N) .....	App. 8.2.2
1109	$H$	Overall height of rectangular HSS main member,	
1110		measured in the plane of the connection, in. (mm).....	
1111		.....	Table D3.1, K2.1
1112	$H$	Flexural constant .....	E4
1113	$H_b$	Overall height of rectangular HSS branch member,	
1114		measured in the plane of the connection, in. (mm).....	K2.1
1115	$H_{bi}$	Overall depth of the overlapping branch .....	K2.3
1116	$I$	Moment of inertia in the plane of bending, in. <sup>4</sup> (mm <sup>4</sup> ).....	App. 8.2.1
1117	$I_c$	Moment of inertia of the concrete section, in. <sup>4</sup> (mm <sup>4</sup> ).....	I2.1
1118	$I_d$	Moment of inertia of the steel deck supported on	
1119		secondary members, in. <sup>4</sup> (mm <sup>4</sup> ).....	App. 2.1
1120	$I_p$	Moment of inertia of primary members, in. <sup>4</sup> (mm <sup>4</sup> ).....	App. 2.1
1121	$I_s$	Moment of inertia of secondary members, in. <sup>4</sup> (mm <sup>4</sup> ).....	App. 2.1
1122	$I_s$	Moment of inertia of steel shape, in. <sup>4</sup> (mm <sup>4</sup> ) .....	I2.1
1123	$I_{sr}$	Moment of inertia of reinforcing bars, in. <sup>4</sup> (mm <sup>4</sup> ) .....	I2.1
1124	$I_x, I_y$	Moment of inertia about the principal axes, in. <sup>4</sup> (mm <sup>4</sup> ).....	E4
1125	$I_y$	Out-of-plane moment of inertia, in. <sup>4</sup> (mm <sup>4</sup> ).....	App. 6.2
1126	$I_z$	Minor principal axis moment of inertia, in. <sup>4</sup> (mm <sup>4</sup> ).....	F10.2
1127	$I_{yc}$	Moment of inertia of the compression flange about y-	
1128		axis, in. <sup>4</sup> (mm <sup>4</sup> ) .....	F4.2
1129	$J$	Torsional constant, in. <sup>4</sup> (mm <sup>4</sup> ) .....	E4
1130	$K$	Effective length factor.....	C3
1131	$K_x$	= Effective length factor for flexural buckling about x-axis	E4
1132	$K_y$	= Effective length factor for flexural buckling about y-axis	E4
1133	$K_z$	Effective length factor for torsional buckling.....	E4
1134	$K_1$	Effective length factor in the plane of bending,	
1135		calculated based on the assumption of no lateral	
1136		translation set equal to 1.0 unless analysis indicates	
1137		that a smaller value may be used .....	App. 8.2.1
1138	$L$	Story height, in. (mm).....	App. 7.3.2
1139	$L$	Length of member, in. (mm).....	H3
1140	$L$	Nominal occupancy live load.....	App. 4.1.4
1141	$L$	Laterally unbraced length of a member, in. (mm) .....	E2
1142	$L$	Span length, in. (mm).....	App. 6.3.2
1143	$L$	Length of member between work points at truss chord	
1144		centerlines, in. (mm) .....	E5

1145	$L_b$	Length between points that are either braced against lateral displacement of compression flange or braced against twist of the cross section, in. (mm) .....	F2
1146			
1147			
1148	$L_b$	Distance between braces, in. (mm) .....	App. 6.2
1149	$L_p$	Limiting laterally unbraced length for the limit state of yielding in. (mm) .....	F2.2
1150			
1151	$L_p$	Column spacing in direction of girder, ft (m) .....	App. 2
1152	$L_{pd}$	Limiting laterally unbraced length for plastic analysis, in. (mm) .....	App. 1.7
1153			
1154	$L_r$	Limiting laterally unbraced length for the limit state of inelastic lateral-torsional buckling, in. (mm) .....	F2.2
1155			
1156	$L_s$	Column spacing perpendicular to direction of girder, ft (m) .....	App. 2.1
1157			
1158	$L_v$	Distance from maximum to zero shear force, in. (mm) .....	G6
1159	$M_A$	Absolute value of moment at quarter point of the unbraced segment, kip-in. (N-mm) .....	F1
1160			
1161	$M_a$	Required flexural strength using ASD load combinations, kip-in. (N-mm) .....	J10.4
1162			
1163	$M_B$	Absolute value of moment at centerline of the unbraced segment, kip-in. (N-mm) .....	F1
1164			
1165	$M_C$	Absolute value of moment at three-quarter point of the unbraced segment, kip-in. (N-mm) .....	F1
1166			
1167	$M_{c(x,y)}$	Available flexural strength determined in accordance with Chapter F, kip-in. (N-mm) .....	H1.1
1168			
1169	$M_{cx}$	Available lateral-torsional strength for strong axis flexure determined in accordance with Chapter F, kip-in. (N-mm) .....	H1.3
1170			
1171			
1172	$M_e$	Elastic lateral-torsional buckling moment, kip-in. (N-mm) .....	F10.2
1173			
1174	$M_{lt}$	First-order moment under LRFD or ASD load combinations caused by lateral translation of the frame only, kip-in. (N-mm) .....	App. 8.2
1175			
1176			
1177	$M_{max}$	Absolute value of maximum moment in the unbraced segment, kip-in. (N-mm) .....	F1
1178			
1179	$M_{mid}$	Major-axis bending moment at the middle of the unbraced segment, kip-in. (N-mm) .....	App. 1.2
1180			
1181	$M_n$	Nominal flexural strength, kip-in. (N-mm) .....	F1
1182	$M_{nt}$	First-order moment using LRFD or ASD load combinations assuming there is no lateral translation of the frame, kip-in. (N-mm) .....	App. 8.2
1183			
1184			
1185	$M_p$	Plastic bending moment, kip-in. (N-mm) .....	Table B4.1
1186	$M_r$	Required second-order flexural strength under LRFD or ASD load combinations, kip-in. (N-mm) .....	App. 8.2
1187			
1188	$M_r$	Required flexural strength using LRFD or ASD load combinations, kip-in. (N-mm) .....	H1.1
1189			
1190	$M_r$	Required flexural strength in chord using LRFD or ASD load combinations, kip-in. (N-mm) .....	K2.2
1191			
1192	$M_{rb}$	Required bracing moment using LRFD or ASD load combinations, kip-in. (N-mm) .....	App. 6.3.2
1193			
1194	$M_{r-ip}$	Required in-plane flexural strength in branch using	

1195		LRFD or ASD load combinations, kip-in. (N-mm).....	K3.2
1196	$M_{r-op}$	Required out-of-plane flexural strength in branch	
1197		using LRFD or ASD load combinations, kip-in. (N-	
1198		mm).....	K3.2
1199	$M_u$	Design flexural strength, using LRFD load	
1200		combinations, kip-in. (N-mm) .....	F1
1201	$M_y$	Yield moment about the axis of bending, kip-in. (N-	
1202		mm).....	Table B4.1
1203	$M_{yc}$	Yield moment in the compression flange .....	F4.2
1204	$M_0$	Smaller major-axis bending moment at the end of the	
1205		unbraced segment, kip-in. (N-mm).....	App. 1.2
1206	$M_1$	Smaller moment, calculated from a first-order analysis,	
1207		at the ends of the portion of the member unbraced in	
1208		the plane of bending under consideration, kip-in. (N-	
1209		mm) App. 1.3	
1210	$M_2$	Larger moment, calculated from a first-order analysis,	
1211		at the ends of the portion of the member unbraced in	
1212		the plane of bending under consideration, kip-in. (N-	
1213		mm) .....	App. 1.3
1214	$N_i$	Notional lateral load applied at level $i$ , kips (N) .....	C2.2
1215	$N_i$	Additional lateral load.....	App. 7.3
1216	$O_v$	Overlap connection coefficient.....	K2.2
1217	$P_b$	Nominal bond strength.....	I5.2
1218	$P_c$	Available axial strength, kips (N) .....	H1.1
1219	$P_c$	Available tensile strength, kips (N) .....	H1.2
1220	$P_{co}$	Available compressive strength out of the plane of	
1221		bending, kip (N) .....	H1.3
1222	$P_e$	Elastic critical buckling load.....	I2.1b
1223	$P_{e1}, P_{e2}$	Elastic critical buckling load for braced and unbraced	
1224		frame, respectively, kips (N) .....	App. 8.2
1225	$P_{l(t, c)}$	First-order axial force using LRFD or ASD load	
1226		combinations as a result of lateral translation of the	
1227		frame only (tension or compression), kips (N).....	App. 8.2
1228	$P_{n(t, c)}$	First-order axial force using LRFD or ASD load	
1229		combinations, assuming there is no lateral transla-	
1230		tion of the frame (tension or compression), kips (N).....	App. 8.2
1231	$P_n$	Nominal axial strength, kips (N).....	D2
1232	$P_{no}$	Nominal axial compressive strength without	
1233		consideration of length effects, kips (N).....	I2.1
1234	$P_{nf}$	Vertical load in columns that are part of moment	
1235		frames in the direction of translation being consid-	
1236		ered, kips (N) .....	App. 8.2.2
1237	$P_p$	Nominal bearing strength.....	J8
1238	$P_r$	Required second-order axial strength using LRFD or	
1239		ASD load combinations, kips (N).....	App. 8.2
1240	$P_r$	Required axial compressive strength using LRFD or	
1241		ASD load combinations, kips (N).....	C2.3
1242	$P_r$	Required axial strength using LRFD or ASD load	
1243		combinations, kips (N) .....	H1.2
1244	$P_r$	Required strength using LRFD or ASD load	

1245		combinations, kips (N) .....	J10.6
1246	$P_r$	Required strength in branch using LRFD or ASD load	
1247		combinations, kips (N) .....	K3.2d
1248	$P_r$	Required axial strength in chord, kips (N) .....	K2.2
1249	$P_{rb}$	Required brace strength using LRFD or ASD load	
1250		combinations, kips (N) .....	App. 6.2
1251	$P_u$	Required axial strength in compression, kips (N) .....	App. 1.4
1252	$P_y$	Axial <i>yield strength</i> , kips (N) .....	C2.3
1253	$Q$	Net reduction factor accounting for all slender	
1254		compression elements .....	E7
1255	$Q_a$	Reduction factor for slender stiffened compression	
1256		elements .....	E7.2
1257	$Q_{ct}$	Available tensile strength, kips (N) .....	I7.3c
1258	$Q_{cv}$	Available shear strength, kips (N) .....	I7.3c
1259	$Q_f$	Chord-stress interaction parameter .....	K2.2
1260	$Q_n$	Nominal strength of one stud shear connector, kips (N) .....	I3.2
1261	$Q_{nt}$	Nominal tensile strength of one stud anchor .....	I7.3b
1262	$Q_{nv}$	Nominal shear strength of one stud anchor .....	I7.3a
1263	$Q_{rt}$	Required tensile strength, kips (N) .....	I7.3c
1264	$Q_{rv}$	Required shear strength, kips (N) .....	I7.3c
1265	$Q_s$	Reduction factor for slender unstiffened compression	
1266		elements .....	E7.1
1267	$R$	Nominal load due to rainwater or snow, exclusive of	
1268		the ponding contribution, ksi (MPa) .....	App. 2.2
1269	$R$	Seismic response modification coefficient .....	A1.1
1270	$R_a$	Required strength using ASD load combinations .....	B3.4
1271	$R_{FIL}$	Reduction factor for joints using a pair of transverse	
1272		fillet welds only .....	App. 3.3
1273	$R_g$	Coefficient to account for group effect .....	I7.2
1274	$R_m$	Factor in Equation A-8-2-6 dependent on type of	
1275		system .....	App. 8.2.2
1276	$R_n$	Nominal strength, specified in Chapters B through K .....	B3.3
1277	$R_n$	Nominal slip resistance, kips (N) .....	J3.8
1278	$R_{nwl}$	Total nominal strength of longitudinally loaded fillet	
1279		welds, as determined in accordance with Table J2.5 .....	J2.4
1280	$R_{nwt}$	Total nominal strength of transversely loaded fillet	
1281		welds, as determined in accordance with Table J2.5	
1282		without the alternate in Section J2.4 (a) .....	J2.4
1283	$R_{nx}$	Horizontal component of the nominal strength of a	
1284		weld group, kips (N) .....	J2.4
1285	$R_{ny}$	Vertical component of the nominal strength of a weld	
1286		group, kips (N) .....	J2.4
1287	$R_p$	Position effect factor for shear studs .....	I7.2
1288	$R_{pc}$	Web plastification factor .....	F4.1
1289	$R_{pg}$	Bending strength reduction factor .....	F5.2
1290	$R_{PJP}$	Reduction factor for reinforced or nonreinforced	
1291		transverse partial-joint-penetration (PJP) groove welds	
1292		.....	App. 3.3
1293	$R_{pt}$	Web plastification factor corresponding to the tension	
1294		flange yielding limit state .....	F4.4

1295	$R_u$	Required strength using LRFD load combinations .....	B3.3
1296	$S$	Elastic section modulus of round HSS, in. <sup>3</sup> (mm <sup>3</sup> ) .....	F8.2
1297	$S$	Lowest elastic section modulus relative to the axis of	
1298		bending, in. <sup>3</sup> (mm <sup>3</sup> ) .....	F12
1299	$S$	Spacing of secondary members, ft (m) .....	App. 2.1
1300	$S$	Nominal snow load .....	App. 4.1.4
1301	$S$	Chord elastic section modulus, in. <sup>3</sup> (mm <sup>3</sup> ) .....	K2.2
1302	$S_c$	Elastic section modulus to the toe in compression	
1303		relative to the axis of bending, in. <sup>3</sup> (mm <sup>3</sup> ) .....	F10.3
1304	$S_e$	Effective section modulus about major axis, in. <sup>3</sup> (mm <sup>3</sup> ) .....	F7.2
1305	$S_{ip}$	Weld effective elastic section modulus per unit length for in-plane	
1306		bending (Table K4.1), in. <sup>2</sup> (mm <sup>2</sup> ) .....	K4
1307	$S_{op}$	Weld effective elastic section modulus per unit length for out-of-	
1308		plane bending (Table K4.1), in. <sup>2</sup> (mm <sup>2</sup> ) .....	K4
1309	$S_{xt}$ , $S_{xc}$	Elastic section modulus referred to tension and	
1310		compression flanges, respectively, in. <sup>3</sup> (mm <sup>3</sup> ) .....	Table B4.1
1311	$S_x$ , $S_y$	Elastic section modulus taken about the principal axes,	
1312		in. <sup>3</sup> (mm <sup>3</sup> ) .....	F2.2, F6
1313	$T$	Nominal forces and deformations due to the design-basis fire	
1314		defined in Section 4.2.1 .....	App. 4.1.4
1315	$T_a$	Required tension force using ASD load combinations, kips (kN) .....	J3.9
1316	$T_b$	Minimum fastener tension given in Table J3.1 or	
1317		J3.1M, kips (kN) .....	J3.8
1318	$T_c$	Available torsional strength, kip-in. (N-mm) .....	H3.2
1319	$T_n$	Nominal torsional strength, kip-in. (N-mm) .....	H3.1
1320	$T_r$	Required torsional strength using LRFD or ASD load	
1321		combinations, kip-in. (N-mm) .....	H3.2
1322	$T_u$	Required tension force using LRFD load	
1323		combinations, kips (kN) .....	J3.9
1324	$U$	Shear lag factor .....	D3.3
1325	$U$	Utilization ratio .....	K2.2
1326	$U_{bs}$	Reduction coefficient, used in calculating block shear	
1327		rupture strength .....	J4.3
1328	$U_p$	Stress index for primary members .....	App. 2.2
1329	$U_s$	Stress index for secondary members .....	App. 2.2
1330	$V'_r$	Required shear force transferred by shear connectors	
1331		using LRFD or ASD load combinations, kips (N) .....	I2.1
1332	$V_c$	Available shear strength, kips (N) .....	G3.3
1333	$V_n$	Nominal shear strength, kips (N) .....	G1
1334	$V_r$	Required shear strength at the location of the stiffener	
1335		using LRFD or ASD load combinations, kips (N) .....	G3.3
1336	$V_r$	Required shear strength using LRFD or ASD load	
1337		combinations, kips (N) .....	H3.2
1338	$Y_i$	Gravity load, applied at the level $i$ , from the LRFD	
1339		load combination or 1.6 times the ASD load combi-	
1340		nation, kips (N) .....	C2.2
1341	$Y_t$	Hole reduction coefficient, kips (N) .....	F13.1
1342	$Z$	Plastic section modulus about the axis of bending, in. <sup>3</sup>	
1343		(mm <sup>3</sup> ) .....	F7.1
1344	$Z_b$	Plastic section modulus of branch about the axis of	



1345	bending, in. <sup>3</sup> (mm <sup>3</sup> ).....	K3.3
1346	$Z_{x,y}$ Plastic section modulus about the principal axes, in. <sup>3</sup>	
1347	(mm <sup>3</sup> ) .....	F2, F6.1
1348	$a$ Clear distance between transverse stiffeners, in. (mm).....	F13.2
1349	$a$ Distance between connectors in a built-up member, in.	
1350	(mm) .....	E6.1
1351	$a$ Shortest distance from edge of pin hole to edge of	
1352	member measured parallel to direction of force, in.	
1353	(mm) .....	D5.1
1354	$a$ Half the length of the nonwelded root face in the	
1355	direction of the thickness of the tension-loaded plate,	
1356	in. (mm) App. 3.3	
1357	$a'$ Weld length along both edges of the cover plate	
1358	termination to the beam or girder .....	F13.3
1359	$a_w$ Ratio of two times the web area in compression due to	
1360	application of major axis bending moment alone to	
1361	the area of the compression flange components .....	F4.2
1362	$b$ Outside width of leg in compression, in. (mm).....	F10.3
1363	$b$ For flanges of I-shaped members, half the full-flange	
1364	width, $b_f$ ; for flanges of channels, the full nominal	
1365	dimension of the flange, in. (mm) .....	F6.2
1366	$b$ Full width of longest angle leg, in. (mm).....	E7.1
1367	$b$ Width of unstiffened compression element; for flanges	
1368	of I-shaped members and tees, the width $b$ is half the	
1369	full-flange width, $b_f$ ; for legs of angles and flanges	
1370	of channels and zees, the width $b$ is the full nominal	
1371	dimension; for plates, the width $b$ is the distance	
1372	from the free edge to the first row of fasteners or line	
1373	of welds, or the distance between adjacent lines of	
1374	fasteners or lines of welds; for rectangular HSS, the	
1375	width $b$ is the clear distance between the webs less	
1376	the inside corner radius on each side, in. (mm) .....	B4.1, B4.2
1377	$b$ Width of the angle leg resisting the shear force, in.	
1378	(mm) .....	G4
1379	$b_{cf}$ Width of column flange, in. (mm) .....	J10.6
1380	$b_e$ Reduced effective width, in. (mm).....	E7.2
1381	$b_e$ Effective edge distance; the distance from the edge of	
1382	the hole to the edge of the part measured in the	
1383	direction normal to the applied force, in. (mm) .....	D5.1
1384	$b_{eoi}$ Effective width of the branch face welded to the chord ....	K2.3
1385	$b_{eov}$ Effective width of the branch face welded to the	
1386	overlapped brace .....	K2.3
1387	$b_f$ Width of flange, in. (mm) .....	B4.1
1388	$b_{fc}$ Width of compression flange, in. (mm) .....	F4.2
1389	$b_{ft}$ Width of tension flange, in. (mm).....	G3.1
1390	$b_l$ Length of longer leg of angle, in. (mm).....	E5
1391	$b_s$ Length of shorter leg of angle, in. (mm).....	E5
1392	$b_s$ Stiffener width for one-sided stiffeners, in. (mm).....	App. 6.3.2
1393	$c$ 2.2	
1394	$d$ Nominal fastener diameter, in. (mm) .....	J3.3

1395	$d$	Nominal bolt diameter, in. (mm) .....	J3.10
1396	$d$	Full nominal depth of the section, in. (mm).....	B4.1
1397	$d$	Full depth of tee, in. (mm) .....	E7.1
1398	$d$	Depth of rectangular bar, in. (mm) .....	F11.2
1399	$d$	Diameter, in. (mm).....	J7
1400	$d$	Diameter of pin, in. (mm) .....	D5.1
1401	$d$	Diameter of roller, in. (mm).....	J7
1402	$d_b$	Depth of beam, in. (mm).....	J10.6
1403	$d_b$	Nominal diameter (body or shank diameter), in. (mm) .....	App. 3.4
1404	$d_c$	Depth of column, in. (mm) .....	J10.6
1405	$e$	Eccentricity in a truss connection, positive being away	
1406		from the branches, in. (mm) .....	K2.1
1407	$e_{mid-ht}$	Distance from the edge of stud shank to the steel deck	
1408		web, measured at mid-height of the deck rib, and in	
1409		the load bearing direction of the stud (in other	
1410		words, in the direction of maximum moment for a	
1411		simply supported beam), in. (mm).....	I7.2
1412	$f'_c$	Specified minimum compressive strength of concrete,	
1413		ksi (MPa) .....	I1.2
1414	$f'_{cm}$	Specified minimum compressive strength of concrete	
1415		at elevated temperatures, ksi (MPa).....	App. 4.2.3.2
1416	$f_o$	Stress due to D + R (the nominal dead load + the	
1417		nominal load due to rainwater or snow exclusive of	
1418		the ponding contribution), ksi (MPa) .....	App. 2.2
1419	$f_{ra}$	Required axial stress at the point of consideration	
1420		using LRFD or ASD load combinations, ksi (MPa).....	H2
1421	$f_{rb(w,z)}$	Required flexural stress at the point of consideration	
1422		(major axis, minor axis) using LRFD or ASD load	
1423		combinations, ksi (MPa).....	H2
1424	$f_{rv}$	Required shear stress using LRFD or ASD load	
1425		combinations, ksi (MPa).....	J3.7
1426	$g$	Transverse center-to-center spacing (gage) between	
1427		fastener gage lines, in. (mm).....	B4.3
1428	$g$	Gap between toes of branch members in a gapped K-	
1429		connection, neglecting the welds, in. (mm).....	K2.1
1430	$h$	Clear distance between flanges less the fillet or corner	
1431		radius at each flange for rolled shapes; for built-up	
1432		sections, the distance between adjacent lines of	
1433		fasteners or the clear distance between flanges when	
1434		welds are used; for tees, the overall depth; for rec-	
1435		tangular HSS, the clear distance between the flanges	
1436		less the inside corner radius on each side, in. (mm) .....	B4.2
1437	$h_c$	Twice the distance from the centroid to the following:	
1438		the inside face of the compression flange less the	
1439		fillet or corner radius, for rolled shapes; the nearest	
1440		line of fasteners at the compression flange or the	
1441		inside faces of the compression flange when welds	
1442		are used, for built-up sections, in. (mm).....	B4.2
1443	$h_f$	Factor for filler .....	J3.8
1444	$h_o$	Distance between flange centroids, in. (mm).....	E4

1445	$h_p$	Twice the distance from the plastic neutral axis to the	
1446		nearest line of fasteners at the compression flange or	
1447		the inside face of the compression flange when	
1448		welds are used, in. (mm).....	B4.2
1449	$j$	Factor defined by Equation G2-6 for minimum	
1450		moment of inertia for a transverse stiffener.....	G2.2
1451	$k$	Distance from outer face of flange to the web toe of	
1452		fillet, in. (mm).....	J10.2
1453	$k_c$	Coefficient for slender unstiffened elements, in. (mm) .....	Table B4.1
1454	$k_{sc}$	Slip-critical combined tension and shear coefficient .....	J3.9
1455	$k_v$	Web plate shear buckling coefficient .....	G2.1
1456	$l$	Actual length of end-loaded weld, in. (mm) .....	J2.2
1457	$l$	Largest laterally unbraced length along either flange at	
1458		the point of load, in. (mm).....	J10.4
1459	$l$	Length of bearing, in. (mm).....	J7
1460	$l$	Length of connection in the direction of loading, in.	
1461		(mm) .....	Table D3.1
1462	$l_b$	Length of bearing (not less than $k$ for end beam	
1463		reactions), in. (mm).....	J10.2
1464	$l_b$	Bearing length of the load, measured parallel to the	
1465		axis of the HSS member, (or measured across the	
1466		width of the HSS in the case of the loaded cap	
1467		plates), in. (mm).....	K1.1
1468	$l_c$	Length of channel shear connector, in. (mm) .....	I7.2
1469	$l_c$	Clear distance, in the direction of the force, between	
1470		the edge of the hole and the edge of the adjacent	
1471		hole or edge of the material, in. (mm) .....	J3.10
1472	$l_e$	Total effective weld length of groove and fillet welds to	
1473		rectangular HSS, for weld strength calculations, in. (mm).....	K4
1474	$l_{ov}$	Overlap length measured along the connecting face of	
1475		the chord beneath the two branches .....	K2.1
1476	$l_p$	Projected length of the overlapping branch on the	
1477		chord .....	K2.1
1478	$n$	Number of nodal braced points within the span .....	App. 6.3
1479	$n$	Threads per inch (per mm) .....	App. 3.4
1480	$n_b$	Number of bolts carrying the applied tension.....	J3.9
1481	$n_s$	Number of slip planes required for the connection to	
1482		slip.....	J3.8
1483	$n_{SR}$	Number of stress range fluctuations in design life.....	App. 3.3
1484	$p$	Pitch, in. per thread (mm per thread) .....	App. 3.4
1485	$p_i$	Ratio of element $i$ deformation to its deformation at	
1486		maximum stress .....	J2.4
1487	$r$	Radius of gyration, in. (mm).....	E2
1488	$r_{crit}$	Distance from instantaneous center of rotation to weld	
1489		element with minimum $\Delta_u/r_i$ ratio, in. (mm).....	J2.4
1490	$r_i$	Minimum radius of gyration of individual component	
1491		in a built-up member, in. (mm).....	E6.1
1492	$r_i$	Distance from instantaneous center of rotation to $i^{th}$ weld	
1493		element, in. (mm) .....	J2.4
1494	$r_o$	Polar radius of gyration about the shear center, in.	

1495	(mm)	.....	E4
1496	$r_t$	Radius of gyration of the flange components in	
1497		flexural compression plus one-third of the web area in	
1498		compression due to application of major axis bending	
1499		moment alone	.....F4.2
1500	$r_{ts}$	Effective radius of gyration used in the determination	
1501		of $L_r$ for the lateral-torsional buckling limit state for	
1502		major axis bending of doubly-symmetric compact I-	
1503		shaped members and channels	.....F2.2
1504	$r_x$	Radius of gyration about geometric axis parallel to	
1505		connected leg, in. (mm)	.....E5
1506	$r_y$	Radius of gyration about y-axis, in. (mm)	.....E4
1507	$r_z$	Radius of gyration about the minor principal axis, in.	
1508		(mm)	.....E5
1509	$s$	Longitudinal center-to-center spacing (pitch) of any	
1510		two consecutive holes, in. (mm)	.....B4.3
1511	$t$	Thickness of element, in. (mm)	.....B4.2
1512	$t$	Thickness of wall, in. (mm)	.....E7.2
1513	$t$	Thickness of angle leg, in. (mm)	.....F10.2
1514	$t$	Width of rectangular bar parallel to axis of bending,	
1515		in. (mm)	.....F11.2
1516	$t$	Thickness of connected material, in. (mm)	.....J3.10
1517	$t$	Thickness of plate, in. (mm)	.....D5.1
1518	$t$	Design wall thickness for HSS equal to 0.93 times the	
1519		nominal wall thickness for ERW HSS and equal to the	
1520		nominal wall thickness for SAW HSS, in. (mm)	.....B4.2
1521	$t$	Total thickness of fillers, in. (mm)	.....J5
1522	$t$	Design wall thickness of HSS main member, in. (mm)	.....K1.1
1523	$t_b$	Design wall thickness of HSS branch member, in.	
1524		(mm)	.....K2.1
1525	$t_{bi}$	Thickness of the overlapping branch, in. (mm)	.....K2.3
1526	$t_{bj}$	Thickness of the overlapped branch, in. (mm)	.....K2.3
1527	$t_{cf}$	Thickness of the column flange, in. (mm)	.....J10.6
1528	$t_f$	Thickness of the loaded flange, in. (mm)	.....J10.1
1529	$t_f$	Thickness of channel anchor flange, in. (mm)	.....I7.2
1530	$t_{fc}$	Thickness of compression flange, in. (mm)	.....F4.2
1531	$t_p$	Thickness of plate, in. (mm)	.....K1.1
1532	$t_p$	Thickness of tension loaded plate, in. (mm)	.....App. 3.3
1533	$t_{st}$	Thickness of web stiffener, in. (mm)	.....App. 6.3.2
1534	$t_w$	Thickness of channel anchor web, in. (mm)	.....I7.2
1535	$t_w$	Thickness of beam web, in. (mm)	.....App. 6.3.2
1536	$t_w$	Thickness of web, in. (mm)	.....Table B4.1
1537	$t_w$	Web thickness, in. (mm)	.....J10.2
1538	$t_w$	Thickness of column web, in. (mm)	.....J10.6
1539	$w$	Width of cover plate, in. (mm)	.....F13.3
1540	$w$	Size of weld leg, in. (mm)	.....J2.2
1541	$w$	Subscript relating symbol to major principal axis	
1542		bending	.....H2
1543	$w$	Width of plate, in. (mm)	.....Table D3.1
1544	$w$	Leg size of the reinforcing or contouring fillet, if any,	

1545		in the direction of the thickness of the tension-loaded	
1546		plate, in. (mm) .....	App. 3.3
1547	$w_c$	Weight of concrete per unit volume ( $90 \leq w_c \leq 155$ lbs / ft <sup>3</sup>	
1548		or $1500 \leq w_c \leq 2500$ kg/m <sup>3</sup> ) .....	I2.1
1549	$w_r$	Average width of concrete rib or haunch, in. (mm).....	I3.2
1550	$x$	Subscript relating symbol to strong axis bending .....	H1.1
1551	$x_i$	x component of $r_i$ .....	J2.4
1552	$y_i$	y component of $r_i$ .....	J2.4
1553	$x_o, y_o$	Coordinates of the shear center with respect to the	
1554		centroid, in. (mm) .....	E4
1555	$\bar{x}$	Connection eccentricity, in. (mm).....	Table D3.1
1556	$y$	Subscript relating symbol to weak axis bending.....	H1.1
1557	$z$	Subscript relating symbol to minor principal axis	
1558		bending .....	H2
1559	$\beta$	Reduction factor given by Equation J2-1.....	J2.2
1560	$\beta$	Width ratio; the ratio of branch diameter to chord	
1561		diameter for round HSS; the ratio of overall branch	
1562		width to chord width for rectangular HSS .....	K2.1
1563	$\beta_T$	Brace stiffness requirement excluding web distortion,	
1564		kip-in./radian (N-mm/radian) .....	App. 6.3.2
1565	$\beta_{br}$	Required brace stiffness .....	App. 6.2.1
1566	$\beta_{eff}$	Effective width ratio; the sum of the perimeters of the	
1567		two branch members in a K-connection divided by	
1568		eight times the chord width.....	K2.1
1569	$\beta_{eop}$	Effective outside punching parameter.....	K2.3
1570	$\beta_{sec}$	Web distortional stiffness, including the effect of web	
1571		transverse stiffeners, if any, kip-in./radian (N-	
1572		mm/radian) .....	App. 6.3.2
1573	$\beta_{Tb}$	Required stiffness for nodal bracing .....	App. 6.3.2a
1574	$\beta_w$	Section property for unequal leg angles, positive for	
1575		short legs in compression and negative for long legs	
1576		in compression .....	F10.2
1577	$\Delta$	First-order interstory drift due to the LRFD or ASD	
1578		load combinations, in. (mm).....	App. 7.3.2
1579	$\Delta_H$	First-order interstory drift due to lateral forces, in.	
1580		(mm) .....	App. 8.2.2
1581	$\Delta_i$	Deformation of weld elements at intermediate stress	
1582		levels, linearly proportioned to the critical deforma-	
1583		tion based on distance from the instantaneous center	
1584		of rotation, $r_i$ , in. (mm) .....	J2.4
1585	$\Delta_{mi}$	Deformation of weld element at maximum stress, in.	
1586		(mm) .....	J2.4
1587	$\Delta_{ui}$	Deformation of weld element at ultimate stress	
1588		(rupture), usually in element furthest from instanta-	
1589		neous center of rotation, in. (mm) .....	J2.4
1590	$\gamma$	Chord slenderness ratio; the ratio of one-half the	
1591		diameter to the wall thickness for round HSS; the	
1592		ratio of one-half the width to wall thickness for	
1593		rectangular HSS .....	K2.1

1594	$\zeta$	Gap ratio; the ratio of the gap between the branches of	
1595		a gapped K-connection to the width of the chord for	
1596		rectangular HSS .....	K2.1
1597	$\eta$	Load length parameter, applicable only to rectangular	
1598		HSS; the ratio of the length of contact of the branch	
1599		with the chord in the plane of the connection to the	
1600		chord width .....	K2.1
1601	$\lambda$	Slenderness parameter .....	F3.2
1602	$\lambda_p$	Limiting slenderness parameter for compact element .....	B4
1603	$\lambda_{pd}$	Limiting slenderness parameter for plastic design .....	App. 1.2
1604	$\lambda_{pf}$	Limiting slenderness parameter for compact flange .....	F3.2
1605	$\lambda_{pw}$	Limiting slenderness parameter for compact web .....	F4
1606	$\lambda_r$	Limiting slenderness parameter for noncompact	
1607		element .....	B4
1608	$\lambda_{rf}$	Limiting slenderness parameter for noncompact flange .....	F3.2
1609	$\lambda_{rw}$	Limiting slenderness parameter for noncompact web .....	F4.2
1610	$\mu$	Mean slip coefficient for class A or B surfaces, as	
1611		applicable, or as established by tests .....	J3.8
1612	$\phi$	Resistance factor, specified in Chapters B through K .....	B3.3
1613	$\phi_B$	Resistance factor for bearing on concrete .....	I5.1
1614	$\phi_b$	Resistance factor for flexure .....	F1
1615	$\phi_c$	Resistance factor for compression .....	E1
1616	$\phi_c$	Resistance factor for axially loaded composite	
1617		columns .....	I2.1b
1618	$\phi_s$	Resistance factor for stud anchor in tension .....	I7.3b
1619	$\phi_{sf}$	Resistance factor for shear on the failure path .....	D5.1
1620	$\phi_T$	Resistance factor for torsion .....	H3.1
1621	$\phi_t$	Resistance factor for tension .....	D2
1622	$\phi_v$	Resistance factor for shear .....	G1
1623	$\phi_v$	Resistance factor for stud anchor in shear .....	I7.3a
1624	$\Omega$	Safety factor, specified in chapters B through K .....	B3.4
1625	$\Omega_B$	Safety factor for bearing on concrete .....	I5.1
1626	$\Omega_b$	Safety factor for flexure .....	F1
1627	$\Omega_c$	Safety factor for compression .....	E1
1628	$\Omega_c$	Safety factor for axially loaded composite columns .....	I2.1b
1629	$\Omega_s$	Safety factor for stud anchor in tension .....	I7.3b
1630	$\Omega_{sf}$	Safety factor for shear on the failure path .....	D5.1
1631	$\Omega_t$	Safety factor for torsion .....	H3.1
1632	$\Omega_t$	Safety factor for tension .....	D2
1633	$\Omega_v$	Safety factor for shear .....	G1
1634	$\Omega_v$	Safety factor for stud anchor in shear .....	I7.3a
1635	$\rho_{sr}$	Minimum reinforcement ratio for longitudinal	
1636		reinforcing I2.1	
1637	$\theta$	Angle of loading measured from the weld longitudinal	
1638		axis, degrees .....	J2.4
1639	$\theta$	Acute angle between the branch and chord, degrees .....	K2.1
1640	$\theta_t$	Angle of loading measured from the longitudinal	
1641		axis of $i^{th}$ weld element, degrees .....	J2.4
1642	$\varepsilon_{cu}$	Strain corresponding to compressive strength, $f'_c$ .....	App. 4.2

$\tau_b$	Parameter for reduced flexural stiffness using the direct analysis method .....	C2.3
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## GLOSSARY

Terms defined in this Glossary are *italicized* in the Glossary and where they first appear within a section or long paragraph throughout the Specification.

Notes:

- (1) Terms designated with † are common AISI-AISC terms that are coordinated between the two standards development organizations.
- (2) Terms designated with \* are usually qualified by the type of *load effect*, for example, *nominal tensile strength*, *available compressive strength*, *design flexural strength*.
- (3) Terms designated with \*\* are usually qualified by the type of component, for example, web *local buckling*, flange *local bending*.

*Allowable strength\*†*. Nominal strength divided by the safety factor,  $R_n/\Omega$ .

*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*.

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

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

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

*Authority having jurisdiction*. Organization, political subdivision, office or individual charged with the responsibility of administering and enforcing the provisions of the *applicable building code*.

*Available strength\*†*. Design strength or allowable strength, as appropriate.

*Available stress\**. Design stress or allowable stress, as appropriate.

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

*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 connection, *limit state* of shear forces transmitted by the mechanical fastener to the connection elements.

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

*Bearing-type connection*. Bolted *connection* where shear forces are



transmitted by the bolt bearing against the connection elements.

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

**Braced frame†.** 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.** In an *HSS connection*, member that terminates at a *chord member* or *main member*.

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

**Buckling strength.** Strength for *instability limit states*.

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

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

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

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

**Cladding.** Exterior covering of structure.

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

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

**Column.** Structural member that has the primary function of resisting axial force.

**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.

**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.

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

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

- the remaining concrete.
- Concrete crushing. Limit state* of compressive failure in concrete having reached the ultimate strain.
- Concrete haunch.* In a *composite* floor system constructed using a *formed steel deck*, the section of solid concrete that results from stopping the deck on each side of the girder.
- Concrete-encased beam.* Beam totally encased in concrete cast integrally with the slab.
- Connection†.** Combination of structural elements and *joints* used to transmit forces between two or more members.
- Construction documents.* Design drawings, specifications, shop drawings and erection drawings.
- Continuous Inspection (CI).* Full-time observation or inspection of work requiring inspection, with the inspector present in the area where the work is being performed.
- Cope.* Cutout made in a structural member to remove a flange and conform to the shape of an intersecting member.
- Cover plate.* Plate welded or bolted to the flange of a member to increase cross-sectional area, section modulus or moment of inertia.
- Cross connection.* *HSS connection* in which forces in *branch members* or connecting elements transverse to the *main member* are primarily equilibrated by forces in other branch members or connecting elements on the opposite side of the main member.
- Design drawings.* Graphic and pictorial documents showing the design, location and dimensions of the work. These documents generally include plans, elevations, sections, details, schedules, diagrams and notes.
- 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*,  $\phi R_n$ .
- 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 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.* In a *composite* section, mechanism by which

- force is transferred between steel and concrete by bond stress.
- Distortional failure. Limit state* of an *HSS* truss connection based on distortion of a rectangular *HSS* chord member into a rhomboidal shape.
- Distortional stiffness.* Out-of-plane flexural stiffness of web.
- Double curvature.* Deformed shape of a beam with one or more inflection points within the span.
- Double-concentrated forces.* Two equal and opposite forces 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 strength at locations of concentrated forces.
- Drift.* Lateral deflection of structure.
- Effective length factor, K.* Ratio between the *effective length* and the unbraced length of the member.
- Effective length.* Length of an otherwise identical *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*.
- 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.
- End return.* Length of *fillet weld* that continues around a corner in the same plane.
- Engineer of record.* Licensed professional responsible for sealing the *design drawings* and *specifications*.
- 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.
- 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* filled with *structural concrete*.
- Filler metal.* Metal or alloy added in making a welded joint.

- 1884 *Filler*. Plate used to build up the thickness of one component.
- 1885 *Fillet weld reinforcement*. *Fillet welds* added to *groove welds*.
- 1886 *Fillet weld*. Weld of generally triangular cross section made between
- 1887 intersecting surfaces of elements.
- 1888 *Finished surface*. Surfaces fabricated with a roughness height value
- 1889 measured in accordance with ANSI/ASME B46.1 that is equal to or
- 1890 less than 500.
- 1891 *First-order analysis*. *Structural analysis* in which equilibrium
- 1892 conditions are formulated on the undeformed structure; *second-*
- 1893 *order effects* are neglected.
- 1894 *Fitted bearing stiffener*. *Stiffener* used at a support or concentrated *load*
- 1895 that fits tightly against one or both flanges of a *beam* so as to
- 1896 transmit load through bearing.
- 1897 *Flare bevel groove weld*. Weld in a groove formed by a member with a
- 1898 curved surface in contact with a planar member.
- 1899 *Flare V-groove weld*. Weld in a groove formed by two members with
- 1900 curved surfaces.
- 1901 *Flat width*. Nominal width of rectangular *HSS* minus twice the outside
- 1902 corner radius. In the absence of knowledge of the corner radius, the
- 1903 flat width may be taken as the total section width minus three times
- 1904 the thickness.
- 1905 *Flexural buckling*†. Buckling mode in which a compression member
- 1906 deflects laterally without twist or change in cross-sectional shape.
- 1907 *Flexural-torsional buckling*†. Buckling mode in which a compression
- 1908 member bends and twists simultaneously without change in cross-
- 1909 sectional shape.
- 1910 *Force*. Resultant of distribution of stress over a prescribed area.
- 1911 *Formed section*. See *cold-formed steel structural member*.
- 1912 *Formed steel deck*. In *composite* construction, *steel* cold formed into a
- 1913 decking profile used as a permanent concrete form.
- 1914 *Fully restrained moment connection*. Connection capable of transferring
- 1915 moment with negligible rotation between connected members.
- 1916 *Gage*. Transverse center-to-center spacing of *fasteners*.
- 1917 *Gap connection*. *HSS* truss connection with a gap or space on the *chord*
- 1918 face between intersecting *branch members*.
- 1919 *Geometric axis*. Axis parallel to web, flange or angle leg.
- 1920 *Girder filler*. In a *composite* floor system constructed using a *formed*
- 1921 *steel deck*, narrow piece of *sheet steel* used as a fill between the
- 1922 edge of a deck sheet and the flange of a girder.
- 1923 *Girder*. See *Beam*.
- 1924 *Girt*†. Horizontal structural member that supports wall panels and is
- 1925 primarily subjected to bending under horizontal *loads*, such as wind
- 1926 *load*.
- 1927 *Gouge*. Relatively smooth surface groove or cavity resulting from
- 1928 plastic deformation or removal of material.
- 1929 *Gravity axis*. Axis through the center of gravity of a member along its
- 1930 length.
- 1931 *Gravity frame*. Portion of the framing system not included in the *lateral*
- 1932 *load resisting system*.
- 1933 *Gravity load*. *Load* acting in the downward direction, such as that

- 1934 produced by dead and live loads.
- 1935 *Grip (of bolt)*. Thickness of material through which a bolt passes.
- 1936 *Groove weld*. Weld in a groove between connection elements. See also
- 1937 AWS D1.1.
- 1938 *Gusset plate*. Plate element connecting truss members or a strut or brace
- 1939 to a *beam* or *column*.
- 1940 *High-strength bolt*. Fastener supplied in compliance with ASTM A325,
- 1941 A325M, A490, A490M, F1852, F2280 or an alternate fastener as
- 1942 permitted in Section J3.1.
- 1943 *Horizontal shear*. In a *composite beam*, force at the interface between
- 1944 steel and concrete surfaces.
- 1945 *HSS*. Square, rectangular or round hollow structural steel section
- 1946 produced in accordance with a *pipe* or tubing product specification.
- 1947 *Inelastic analysis*. *Structural analysis* that takes into account inelastic
- 1948 material behavior, including *plastic analysis*.
- 1949 *In-plane instability*†. *Limit state* involving buckling in the plane of the
- 1950 frame or the member.
- 1951 *Instability*†. *Limit state* reached in the loading of a structural
- 1952 component, frame or structure in which a slight disturbance in the
- 1953 loads or geometry produces large displacements.
- 1954 *Introduction length*. Length along an *encased composite column* in
- 1955 which the column force is assumed to be transferred into or out of
- 1956 the steel shape
- 1957 *Joint eccentricity*. In an *HSS truss connection*, perpendicular distance
- 1958 from *chord member* center of gravity to intersection of *branch*
- 1959 *member* work points.
- 1960 *Joint*†. Area where two or more ends, surfaces, or edges are attached.
- 1961 Categorized by type of *fastener* or weld used and method of force
- 1962 transfer.
- 1963 *k-area*. The region of the web that extends from the tangent point of the
- 1964 web and the flange-web fillet (AISC *k* dimension) a distance 1½ in.
- 1965 (38 mm) into the web beyond the *k* dimension.
- 1966 *K-connection*. *HSS connection* in which forces in *branch members* or
- 1967 connecting elements transverse to the *main member* are primarily
- 1968 equilibrated by forces in other branch members or connecting
- 1969 elements on the same side of the main member.
- 1970 *Lacing*. Plate, angle or other steel shape, in a lattice configuration, that
- 1971 connects two steel shapes together.
- 1972 *Lap joint*. Joint between two overlapping connection elements in
- 1973 parallel planes.
- 1974 *Lateral bracing*. Member or system that is designed to inhibit lateral
- 1975 buckling or lateral-torsional buckling of primary framing members.
- 1976 *Lateral load resisting system*. Structural system designed to resist lateral
- 1977 loads and provide stability for the structure as a whole.
- 1978 *Lateral load*. Load acting in a lateral direction, such as that produced by
- 1979 wind or earthquake effects.
- 1980 *Lateral-torsional buckling*†. Buckling mode of a flexural member
- 1981 involving deflection out of the plane of bending occurring simulta-
- 1982 neously with twist about the shear center of the cross-section.
- 1983 *Leaning column*. Column designed to carry *gravity loads* only, with

1984	<i>connections</i> that are not intended to provide resistance to <i>lateral</i>
1985	<i>loads</i> .
1986	<i>Length effects</i> . Consideration of the reduction in strength of a member
1987	based on its <i>unbraced length</i> .
1988	<i>Limit state</i> †. Condition in which a structure or component becomes unfit
1989	for service and is judged either to be no longer useful for its in-
1990	tended function ( <i>serviceability limit state</i> ) or to have reached its
1991	ultimate load-carrying capacity ( <i>strength limit state</i> ).
1992	<i>Load</i> †. Force or other action that results from the weight of building
1993	materials, occupants and their possessions, environmental effects,
1994	differential movement, or restrained dimensional changes.
1995	<i>Load effect</i> †. Forces, stresses and deformations produced in a <i>structural</i>
1996	<i>component</i> by the applied loads.
1997	<i>Load factor</i> †. Factor that accounts for deviations of the <i>nominal load</i>
1998	from the actual <i>load</i> , for uncertainties in the analysis that transforms
1999	the load into a <i>load effect</i> and for the probability that more than one
2000	extreme load will occur simultaneously.
2001	<i>Local bending</i> ** †. <i>Limit state</i> of large deformation of a flange under a
2002	concentrated transverse force.
2003	<i>Local buckling</i> ** †. <i>Limit state</i> of <i>buckling</i> of a compression element
2004	within a cross section.
2005	<i>Local yielding</i> **†. <i>Yielding</i> that occurs in a local area of an element.
2006	<i>LRFD (Load and Resistance Factor Design)</i> †. Method of proportioning
2007	<i>structural components</i> such that the <i>design strength</i> equals or
2008	exceeds the <i>required strength</i> of the component under the action of
2009	the <i>LRFD load combinations</i> .
2010	<i>LRFD load combination</i> †. Load combination in the <i>applicable building</i>
2011	<i>code</i> intended for strength design ( <i>load and resistance factor</i>
2012	<i>design</i> ).
2013	<i>Main member</i> . In an <i>HSS connection</i> , <i>chord member</i> , column or other
2014	<i>HSS member</i> to which <i>branch members</i> or other connecting
2015	elements are attached.
2016	<i>Mechanism</i> . Structural system that includes a sufficient number of real
2017	hinges, plastic hinges or both, so as to be able to articulate in one or
2018	more rigid body modes.
2019	<i>Mill scale</i> . Oxide surface coating on steel formed by the hot rolling
2020	process.
2021	<i>Moment connection</i> . Connection that transmits bending moment
2022	between connected members.
2023	<i>Moment frame</i> †. Framing system that provides resistance to lateral loads
2024	and provides stability to the <i>structural system</i> , primarily by shear
2025	and flexure of the framing members and their connections.
2026	<i>Negative Flexural Strength</i> . Flexural strength of a <i>composite beam</i> in
2027	regions with flexural tension on the top surface.
2028	<i>Net area</i> . Gross area reduced to account for removed material.
2029	<i>Nodal brace</i> . Brace that prevents lateral movement or twist independ-
2030	ently of other braces at adjacent brace points (see <i>relative brace</i> ).
2031	<i>Nominal dimension</i> . Designated or theoretical dimension, as in tables of
2032	section properties.
2033	<i>Nominal load</i> †. Magnitude of the <i>load</i> specified by the <i>applicable</i>

*building code.*

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

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

*Noncompact section.* Section that 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 the integrity of the material or component is not affected.

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

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

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

*Overlap connection.* *HSS truss connection* in which intersecting *branch members* overlap.

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

*Partial-joint-penetration 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.

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

*Periodic Inspection (PI).* Part-time or intermittent observation or inspection of work requiring inspection that is performed prior to, during, or after the work.

*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, that is, that equilibrium is satisfied and the stress is at or below the yield stress throughout the structure.

*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.* In a *composite* member, method for determining stresses assuming that the steel section and the concrete in the cross section are fully plastic.
- Plastification.* In an HSS connection, *limit state* based on an out-of-plane flexural yield line *mechanism* in the *chord* at a *branch member* connection.
- Plate girder.* Built-up *beam*.
- Plug weld.* Weld made in a circular hole in one element of a joint fusing that element to another element.
- Ponding.* Retention of water due solely to the deflection of flat roof framing.
- Positive flexural strength.* Flexural strength of a *composite* beam in regions having flexural compression on the top surface.
- Pretensioned bolt.* Bolt tightened to the specified minimum pretension.
- Pretensioned joint.* Joint with high-strength bolts tightened to the specified minimum pretension.
- Properly developed.* Reinforcing bars detailed to yield in a ductile manner before crushing of the concrete occurs. Bars meeting the provisions of ACI 318 insofar as development length, spacing and cover, shall be deemed to be properly developed.
- Prying action.* Amplification of the tension force in a bolt caused by leverage between the point of applied load, the bolt and the reaction of the connected elements.
- Punching load.* In an HSS connection, component of *branch member* force perpendicular to a *chord*.
- Purlin*<sup>†</sup>. 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.* Monitoring and inspection tasks performed by an agency or firm other than the fabricator or erector to ensure that the material provided and work performed by the fabricator and erector meet the requirements of the approved construction documents and referenced standards. *Quality assurance* includes those tasks designated “special inspection” by the *applicable building code*.
- Quality assurance inspector (QAI).* Individual designated to provide *quality assurance* inspection for the work being performed.
- Quality assurance plan (QAP).* Program in which the agency or firm responsible for *quality assurance* maintains detailed monitoring and inspection procedures to ensure conformance with the approved construction documents and referenced standards.
- Quality control.* Controls and inspections implemented by the fabricator or erector, as applicable, to ensure that the material provided and work performed meet the requirements of the approved construction documents and referenced standards.



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

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

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

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

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

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

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

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

*Second-order 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 response modification factor.* Factor that reduces seismic *load effects* to strength level.

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

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

*Serviceability limit state†.* Limiting condition affecting the ability of a structure to preserve its appearance, maintainability, durability 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 lag.* Non-uniform tensile stress distribution in a member or connecting element in the vicinity of a connection.

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

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

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

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

*Sidesway Buckling (frame) Stability limit state* involving lateral sideway instability of a frame.

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

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

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

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

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

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

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

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

*Specifications.* Written documents containing the requirements for materials, standards and workmanship.

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

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

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

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

*Statically loaded.* Not subject to significant fatigue stresses. Gravity, wind and seismic loadings are considered to be static loadings.

*Steel anchor.* Headed stud or hot rolled channel welded to a steel

member and embodied in concrete of a *composite member* to transmit shear, tension or a combination of shear and tension at the interface of the two materials.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

*Thermally cut*. Cut with gas, plasma or laser.

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

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

2284 *Torsional bracing*. Bracing resisting twist of a *beam* or *column*.  
 2285 *Torsional buckling†*. Buckling mode in which a compression member  
 2286 twists about its shear center axis.  
 2287 *Transverse reinforcement*. In an *encased concrete composite column*,  
 2288 steel reinforcement in the form of closed ties or welded wire fabric  
 2289 providing confinement for the concrete surrounding the steel shape .  
 2290 *Transverse stiffener*. Web *stiffener* oriented perpendicular to the flanges,  
 2291 attached to the web.  
 2292 *Tubing*. See *HSS*.  
 2293 *Turn-of-nut method*. Procedure whereby the specified pretension in  
 2294 high-strength bolts is controlled by rotating the fastener component  
 2295 a predetermined amount after the bolt has been snug tightened.  
 2296 *Unbraced length*. Distance between braced points of a member,  
 2297 measured between the centers of gravity of the bracing members.  
 2298 *Uneven load distribution*. In an *HSS connection*, condition in which the  
 2299 load is not distributed through the cross section of connected  
 2300 elements in a manner that can be readily determined.  
 2301 *Unframed end*. The end of a member not restrained against rotation by  
 2302 stiffeners or connection elements.  
 2303 *Unstiffened element*. Flat compression element with an adjoining out-of-  
 2304 plane element along one edge parallel to the direction of loading.  
 2305 *Variable load†*. Load not classified as *permanent load*.  
 2306 *Vertical bracing system*. System of *shear walls*, *braced frames* or both,  
 2307 extending through one or more floors of a building.  
 2308 *Weak axis*. Minor principal centroidal axis of a cross section.  
 2309 *Weathering steel*. High-strength, low-alloy steel that, with suitable  
 2310 precautions, can be used in normal atmospheric exposures (not  
 2311 marine) without protective paint coating.  
 2312 *Web crippling†*. Limit state of local failure of web plate in the  
 2313 immediate vicinity of a concentrated load or reaction.  
 2314 *Web sidesway buckling*. Limit state of lateral buckling of the tension  
 2315 flange opposite the location of a concentrated compression force.  
 2316 *Weld metal*. Portion of a fusion weld that has been completely melted  
 2317 during welding. Weld metal has elements of filler metal and base  
 2318 metal melted in the weld thermal cycle.  
 2319 *Weld root*. See *root of joint*.  
 2320 *Y-connection*. *HSS connection* in which the *branch member* or  
 2321 connecting element is not perpendicular to the *main member* and in  
 2322 which forces transverse to the main member are primarily equilibri-  
 2323 ated by shear in the main member.  
 2324 *Yield moment†*. In a member subjected to bending, the moment at which  
 2325 the extreme outer fiber first attains the *yield stress*.  
 2326 *Yield point†*. First stress in a material at which an increase in strain  
 2327 occurs without an increase in stress as defined by ASTM.  
 2328 *Yield strength†*. Stress at which a material exhibits a specified limiting  
 2329 deviation from the proportionality of stress to strain as defined by  
 2330 ASTM.  
 2331 *Yield stress†*. Generic term to denote either *yield point* or *yield strength*,  
 2332 as appropriate for the material.  
 2333 *Yielding†*. Limit state of inelastic deformation that occurs when the *yield*

stress is reached.

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

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

FOR COMMITTEE USE ONLY

## CHAPTER A GENERAL PROVISIONS

This chapter states the scope of the Specification, summarizes referenced specification, code, and standard documents, and provides requirements for materials and structural design documents.

The chapter is organized as follows:

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

### A1. SCOPE

The *Specification for Structural Steel Buildings*, hereafter referred to as the Specification, shall apply to the design of the *structural steel* system, where the steel elements are defined in the *AISC Code of Standard Practice for Steel Buildings and Bridges*, Section 2.1.

This Specification includes the Symbols, the Glossary, Chapters A through M, and Appendices 1 through 8. The Commentary and the User Notes interspersed throughout are not part of the Specification.

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

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

Wherever this Specification refers to the *applicable building code* and there is no local building code, the loads, *load combinations*, system limitations, and general design requirements shall be those in SEI/ASCE 7.

Where conditions are not covered by the 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 are permitted, provided such alternate methods or criteria are acceptable to the authority having jurisdiction.

**User Note:** For the design of structural members, other than hollow structural sections (*HSS*), that are cold-formed to shapes, with elements not more than 1 in. (25 mm) in thickness, the provisions in the *AISI North American Specification for the Design of Cold-Formed Steel Structural Members* are recommended.

### 1. Seismic Applications

The *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341) shall apply to the design of seismic force resisting systems of structural steel or of

structural steel acting compositely with reinforced concrete, unless specifically exempted by the applicable building code.

**User Note:** SEI/ASCE 7 (Table 12.2-1, Item H) specifically exempts structural steel systems, but not composite systems, in seismic design categories B and C if they are designed according to the *Specification* and the seismic loads are computed using a seismic response modification factor,  $R$ , of 3. For seismic design category A, SEI/ASCE 7 does specify lateral forces to be used as the seismic loads and effects, but these calculations do not involve the use of an  $R$  factor. Thus for seismic design category A it is not necessary to define a seismic force resisting system that meets any special requirements and the *Seismic Provisions* do not apply.

The provisions of Appendix 1 of this Specification shall not apply to the seismic design of buildings and other structures.

## 2. Nuclear Applications

The design, fabrication and erection of nuclear structures shall comply with the requirements of the *Specification for Safety-Related Steel Structures for Nuclear Facilities* (ANSI/AISC N690), in addition to the provisions of this Specification.

### A2. REFERENCED SPECIFICATIONS, CODES AND STANDARDS

**Comment [A1]:** Excluded from March 2009 public review.

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

#### ACI International (ACI)

ACI 318-02 *Building Code Requirements for Structural Concrete and Commentary*

ACI 318M-02 *Metric Building Code Requirements for Structural Concrete and Commentary*

ACI 349-06 (ADD TITLE)

#### American Institute of Steel Construction, Inc. (AISC)

AISC 303-05 *Code of Standard Practice for Steel Buildings and Bridges*

ANSI/AISC 341-05 *Seismic Provisions for Structural Steel Buildings*

ANSI/AISC N690-1994(R2004) *Specification for the Design, Fabrication and Erection of Steel Safety-Related Structures for Nuclear Facilities*, including Supplement No. 2

ANSI/AISC N690L-03 *Load and Resistance Factor Design Specification for Steel Safety-Related Structures for Nuclear Facilities*

#### American Society of Civil Engineers (ASCE)

SEI/ASCE 7-02 *Minimum Design Loads for Buildings and Other Structures*

ASCE/SFPE 29-99 *Standard Calculation Methods for Structural Fire Protection*

#### American Society of Mechanical Engineers (ASME)

ASME B18.2.6-96 *Fasteners for Use in Structural Applications*

ASME B46.1-95 *Surface Texture, Surface Roughness, Waviness, and Lay*

**ASTM International (ASTM)**

A6/A6M-04a *Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling*

A36/A36M-04 *Standard Specification for Carbon Structural Steel*

A53/A53M-02 *Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless*

A193/A193M-04a *Standard Specification for Alloy-Steel and Stainless Steel Bolting Materials for High-Temperature Service*

A194/A194M-04 *Standard Specification for Carbon and Alloy Steel Nuts for Bolts for High Pressure or High-Temperature Service, or Both*

A216/A216M-93(2003) *Standard Specification for Steel Castings, Carbon, Suitable for Fusion Welding, for High Temperature Service*

A242/A242M-04 *Standard Specification for High-Strength Low-Alloy Structural Steel*

A283/A283M-03 *Standard Specification for Low and Intermediate Tensile Strength Carbon Steel Plates*

A307-03 *Standard Specification for Carbon Steel Bolts and Studs, 60,000 PSI Tensile Strength*

A325-04 *Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength*

A325M-04 *Standard Specification for High-Strength Bolts for Structural Steel Joints (Metric)*

A354-03a *Standard Specification for Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners*

A370-03a *Standard Test Methods and Definitions for Mechanical Testing of Steel Products*

A449-04 *Standard Specification for Quenched and Tempered Steel Bolts and Studs*

A490-04 *Standard Specification for Heat-Treated Steel Structural Bolts, 150 ksi Minimum Tensile Strength*

A490M-04 *Standard Specification for High-Strength Steel Bolts, Classes 10.9 and 10.9.3, for Structural Steel Joints (Metric)*

A500-03a *Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes*

A501-01 *Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing*

A502-03 *Standard Specification for Steel Structural Rivets*

A514/A514M-00a *Standard Specification for High-Yield Strength, Quenched and Tempered Alloy Steel Plate, Suitable for Welding*

A529/A529M-04 *Standard Specification for High-Strength Carbon-Manganese Steel of Structural Quality*

A563-04 *Standard Specification for Carbon and Alloy Steel Nuts*

A563M-03 *Standard Specification for Carbon and Alloy Steel Nuts [Metric]*

A568/A568M-03 *Standard Specification for Steel, Sheet, Carbon, and High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, General Requirements for*

A572/A572M-04 *Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel*

A588/A588M-04 *Standard Specification for High-Strength Low-Alloy Structural Steel with 50 ksi [345 MPa] Minimum Yield Point to 4 in. [100 mm] Thick*

A606-04 *Standard Specification for Steel, Sheet and Strip, High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance*

A618/A618M-04 *Standard Specification for Hot-Formed Welded and*



*Seamless High-Strength Low-Alloy Structural Tubing*  
A673/A673M-04 *Standard Specification for Sampling Procedure for Impact Testing of Structural Steel*  
A668/A668M-04 *Standard Specification for Steel Forgings, Carbon and Alloy, for General Industrial Use*  
A709/A709M-04 *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*  
A751-01 *Standard Test Methods, Practices, and Terminology for Chemical Analysis of Steel Products*  
A847-99a(2003) *Standard Specification for Cold-Formed Welded and Seamless High-Strength, Low-Alloy Structural Tubing with Improved Atmospheric Corrosion Resistance*  
A852/A852M-03 *Standard Specification for Quenched and Tempered Low-Alloy Structural Steel Plate with 70 ksi [485 MPa] Minimum Yield Strength to 4 in. [100 mm] Thick*  
A913/A913M-04 *Standard Specification for High-Strength Low-Alloy Steel Shapes of Structural Quality, Produced by Quenching and Self-Tempering Process (QST)*  
A992/A992M-04 *Standard Specification for Structural Steel Shapes*

**User Note:** ASTM A992 is the most commonly referenced specification for W shapes.

A1011/A1011M-04 *Standard Specification for Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with Improved Formability*  
A1043/A1043M-05 *Standard Specification for Structural Steel with Low Yield to Tensile Ratio for Use in Buildings.*  
C33-03 *Standard Specification for Concrete Aggregates*  
C330-04 *Standard Specification for Lightweight Aggregates for Structural Concrete*  
E119-00a *Standard Test Methods for Fire Tests of Building Construction and Materials*  
E165-02 *Standard Test Method for Liquid Penetrant Examination*  
E709-01 *Standard Guide for Magnetic Particle Examination*  
F436-03 *Standard Specification for Hardened Steel Washers*  
F844-07a *Standard Specification for Washers, Steel Plain (Flat), Unhardened for General Use*  
F959-02 *Standard Specification for Compressible-Washer-Type Direct Tension Indicators for Use with Structural Fasteners*  
F1554-99 *Standard Specification for Anchor Bolts, Steel, 36, 55, and 105 ksi Yield Strength*

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

F1852-04 *Standard Specification for "Twist-Off" Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength*  
F2280-06 *Standard Specification for Twist Off Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 150 ksi Minimum Tensile Strength*

American Welding Society (AWS)

AWS D1.1/D1.1M-2008 *Structural Welding Code–Steel*  
 AWS D1.3 (add title)  
 AWS A5.1-2004 *Specification for Carbon Steel Electrodes for Shielded Metal Arc Welding*  
 AWS A5.5-96 *Specification for Low-Alloy Steel Electrodes for Shielded Metal Arc Welding*  
 AWS A5.17/A5.17M-97 *Specification for Carbon Steel Electrodes and Fluxes for Submerged Arc Welding*  
 AWS A5.18:2001 *Specification for Carbon Steel Electrodes and Rods for Gas Shielded Arc Welding*  
 AWS A5.20-95 *Specification for Carbon Steel Electrodes for Flux Cored Arc Welding*  
 AWS A5.23/A5.23M-97 *Specification for Low-Alloy Steel Electrodes and Fluxes for Submerged Arc Welding*  
 AWS A5.25/A5.25M-97 *Specification for Carbon and Low-Alloy Steel Electrodes and Fluxes for Electroslag Welding*  
 AWS A5.26/A5.26M-97 *Specification for Carbon and Low-Alloy Steel Electrodes for Electrode Gas Welding*  
 AWS A5.28-96 *Specification for Low-Alloy Steel Electrodes and Rods for Gas Shielded Arc Welding*  
 AWS A5.29:1998 *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*

### **A3. MATERIAL**

#### **1. Structural Steel Materials**

Material test reports or reports of tests made by the fabricator or a testing laboratory shall constitute sufficient evidence of conformity with one of the 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.

#### **1a. ASTM Designations**

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

##### **(1) 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  
 ASTM A1043/A1043M

## (2) Structural tubing

ASTM A500  
 ASTM A501  
 ASTM A618  
 ASTM A847

## (3) Pipe

ASTM A53/A53M, Gr. B

## (4) 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  
 ASTM A1043/A1043M

## (5) Bars

ASTM A36/A36M  
 ASTM A529/A529M  
 ASTM A572/A572M  
 ASTM A709/A709M

## (6) Sheets

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

**1b. Unidentified Steel**

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

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

**1c. Rolled Heavy Shapes**

ASTM A6/A6M hot-rolled shapes with a flange thickness exceeding 2 in. (50 mm), 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 structural design documents shall require that such shapes be supplied with *Charpy V-Notch (CVN) impact test* results in accordance with ASTM A6/A6M, Supplementary Requirement S30, *Charpy V-Notch Impact Test for Structural Shapes – Alternate Core Location*. The impact test shall meet a minimum average value of 20 ft-lbs (27 J) absorbed energy at +70 °F (+21 °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 2 in. (50 mm) that have shapes with flange or web elements less than 2 in. (50 mm) thick welded with *complete-joint-penetration groove welds* to the face of the shapes with thicker elements.

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

#### 1d. Built-Up Heavy Shapes

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

The above requirements also apply to built-up cross-sections consisting of plates exceeding 2 in. (50 mm) that are welded with complete-joint-penetration groove welds to the face of other sections.

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

### 2. Steel Castings and Forgings

Steel castings shall conform to ASTM A216/A216M, Grade 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.

### 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 A354  
 ASTM A449  
 ASTM A490  
 ASTM A490M  
 ASTM F1852  
 ASTM F2280

(2) Nuts:

ASTM A194/A194M  
 ASTM A563  
 ASTM A563M

(3) Washers:

ASTM F436  
 ASTM F436M  
 ASTM F844

(4) Compressible-Washer-Type Direct Tension Indicators:

ASTM F959  
 ASTM F959M

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

#### 4. Anchor Rods and Threaded Rods

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

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

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

,  
 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.

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

#### 5. Consumables for Welding

*Filler metals* and fluxes shall conform to one of the following specifications

of the American Welding Society:

AWS A5.1  
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

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Manufacturer's certification shall constitute sufficient evidence of conformity with the standards. Filler metals and fluxes that are suitable for the intended application shall be selected.

#### **6. Stud Shear Connectors**

Steel stud *shear connectors* shall conform to the requirements of *Structural Welding Code – Steel* (AWS D1.1).

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

#### **A4. STRUCTURAL DESIGN DRAWINGS AND SPECIFICATIONS**

The structural design drawings and specifications shall meet the requirements in the *Code of Standard Practice for Steel Buildings and Bridges*.

## CHAPTER B DESIGN REQUIREMENTS

This chapter addresses general requirements for the analysis and design of steel structures applicable to all chapters of the specification.

The chapter is organized as follows:

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

### B1. GENERAL PROVISIONS

The design of members and *connections* shall be consistent with the intended behavior of the framing system and the assumptions made in the *structural analysis*. Unless restricted by the *applicable building code*, *lateral load* resistance and *stability* may be provided by any combination of members and connections.

### B2. LOADS AND LOAD COMBINATIONS

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

**User Note:** For design according to Section B3.3 (LRFD), the load combinations in SEI/ASCE 7, Section 2.3 apply. For design according to Section B3.4 (ASD), the load combinations in SEI/ASCE 7, Section 2.4 apply.

### B3. 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).

#### 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 Section B2.

Design by *elastic*, *inelastic* or *plastic analysis* is permitted. Provisions for

*Specification for Structural Steel Buildings*, Public Review Draft dated March 1, 2009

inelastic and plastic analysis are as stipulated in Appendix 1, Inelastic Analysis and Design.

## 2. Limit States

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

Design criteria for member connections that are required to meet the structural integrity requirements of the *applicable building code* reflect nominal strength performance criteria, rather than the design strength (LRFD) or the allowable strength (ASD) of the Specification.

Limit states for simple shear connections, based on deformation or yielding of the connection components, are not considered limit states for meeting the structural integrity requirements. Rupture limit states for applicable nominal limit states must be checked.

For the purpose of satisfying only the structural integrity provisions of the applicable building code, bearing bolts in connections with short-slotted holes parallel to the direction of the tension load are permitted. For the purpose of checking bearing, these bolts shall be assumed to be located at the end of the slot.

## 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*. All provisions of this Specification, except for those in Section B3.4, shall apply.

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

$$R_u \leq \phi R_n \quad (\text{B3-1})$$

where

$R_u$  = required strength using LRFD load combinations

$R_n$  = *nominal strength*, specified in Chapters B through K

$\phi$  = *resistance factor*, specified in Chapters B through K

$\phi R_n$  = design strength

## 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*. All provisions of this Specification, except those of Section B3.3, shall apply.

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

$$R_a \leq R_n / \Omega \quad (\text{B3-2})$$

where

$R_a$  = required strength using ASD load combinations

$R_n$  = *nominal strength*, specified in Chapters B through K

$\Omega$  = safety factor, specified in Chapters B through K

$R_n / \Omega$  = allowable strength

## 5. Design for Stability

*Stability* of the structure and its elements shall be determined in accordance with Chapter C.

## 6. Design of Connections

*Connection* elements shall be designed in accordance with the provisions of Chapters J and K. 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*. Self-limiting inelastic deformations of the connections are permitted. At points of support, beams, girders and trusses shall be restrained against rotation about their longitudinal axis unless it can be shown by analysis that the restraint is not required.

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

### 6a. Simple Connections

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

### 6b. Moment Connections

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

#### (a) *Fully-Restrained (FR) Moment Connections*

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

## 7. Moment Redistribution in Beams

The required flexural strength of *beams* satisfying the ductility requirements of Section 1.2 of Appendix 1 may be taken as nine-tenths of the negative moments at the points of support, produced by the *gravity loading* and determined by an *elastic analysis* satisfying the requirements of Chapter C, provided that the maximum positive moment is increased by one-tenth of the average elastic analysis negative moments. This reduction is not permitted for moments produced by loading on cantilevers, for design using *partially-restrained (PR) moment connections*, for design by *inelastic analysis* using the provisions of Appendix 1, for the design of *connections*, or for the design of *columns*. This reduction is permitted for design according to Section B3.3 (LRFD) and for design according to Section B3.4 (ASD). The required axial strength in the beam shall not exceed  $0.15\phi_c F_y A_g$  for LRFD or  $0.15F_y A_g / \Omega_c$  for ASD.

For design according to Section B3.4 (ASD), all references to the design strengths,  $\phi R_n$ , shall be replaced by the corresponding allowable strengths,  $R_n / \Omega$ , and all references to the required strengths using LRFD load combinations,  $R_u$ , shall be replaced by the required strengths using ASD load combinations,  $R_a$ , in the ductility requirements of Section 1.2 of Appendix 1.

## 8. Diaphragms and Collectors

*Diaphragms* and *collectors* shall be designed for forces that result from loads as stipulated in Section B2. They shall be designed in conformance with the provisions of Chapters C, D, E, F, G, H, I and J as applicable.

## 9. Design for Serviceability

The overall structure and the individual members, *connections*, and connectors shall be checked for serviceability. Requirements for serviceability design are given in Chapter L.

## 10. 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  $\frac{1}{4}$  in. per ft (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.

See Appendix 2, Design for Ponding, for methods of checking ponding.

## 11. Design for Fatigue

*Fatigue* shall be considered in accordance with Appendix 3, 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.

## 12. Design for Fire Conditions

Two methods of design for fire conditions are provided in Appendix 4, Structural Design for Fire Conditions: by Analysis and by Qualification Testing. Compliance with the fire protection requirements in the *applicable building code* shall be deemed to satisfy the requirements of this section and Appendix 4.

Nothing in this section is intended to create or imply a contractual requirement for the *engineer of record* responsible for the structural design or any other member of the design team.

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

## 13. Design for Corrosion Effects

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

## 14. Anchorage to Concrete

See Chapter I for anchorage between steel and concrete acting compositely. See Chapter J for design of column bases and anchor rods.

## B4. MEMBER PROPERTIES

### 1. Classification of Sections for Local Buckling

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

For flexure, 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 shall not exceed the limiting width-thickness ratios  $\lambda_p$  from Table B4.1b. If the width-thickness ratio of one or more compression elements exceeds  $\lambda_p$ , but does not exceed  $\lambda_r$  from Table B4.1b, the section is noncompact. If the width-thickness ratio of any element exceeds  $\lambda_r$ , the section is a slender-element section.

#### 1a. Unstiffened Elements

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

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

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

#### 1b. Stiffened Elements

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

- (a) For webs of rolled 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.

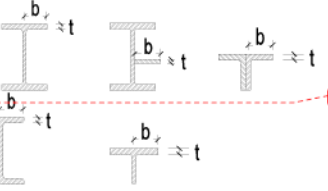
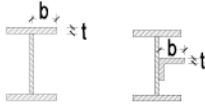
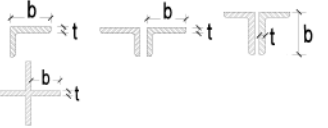

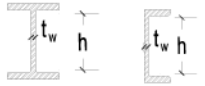




- (d) For flanges of rectangular hollow structural sections (*HSS*), the width  $b$  is the clear distance between webs less the inside corner radius on each side. For webs of rectangular *HSS*,  $h$  is the clear distance between the flanges less the inside corner radius on each side. If the corner radius is not known,  $b$  and  $h$  shall be taken as the corresponding outside dimension minus three times the thickness. For round *HSS*, the diameter,  $D$ , is the chord diameter. The thickness,  $t$ , shall be taken as the *design wall thickness*, per Section B4.2.
- (e) For perforated cover plates,  $b$  is the transverse distance between the nearest line of fasteners, and the net area of the plate is taken at the widest hole.

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

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

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**TABLE B4.1a**

TABLE B4.1a				
Limiting Width-Thickness Ratios for Compression Elements in Members Subject to Axial Compression For Use with Chapter E				
Case	Description of Element	Width Thickness Ratio	Limiting Width-Thickness Ratio $\lambda_r$ (non-slender / slender)	Example
Unstiffened Elements	1 Flanges of rolled I-shaped sections, plates projecting from rolled I-shaped sections; outstanding legs of pairs of angles <del>connected with continuous contact</del> , flanges of channels, and flanges of tees	b/t	$0.56 \sqrt{\frac{E}{F_y}}$	 <del>Deleted: non-compact</del> <del>Deleted: in</del>
	2 Flanges of built-up I-shaped sections and plates or angle legs projecting from built-up I-shaped sections	b/t	$0.64 \sqrt{\frac{k_c E}{F_y}}$ [9]	
	3 Legs of single angles, legs of double angles with separators, and all other unstiffened elements	b/t	$0.45 \sqrt{\frac{E}{F_y}}$	
	4 Stems of tees	d/t	$0.75 \sqrt{\frac{E}{F_y}}$	
	5 Webs of doubly-symmetric I-shaped sections and channels	h/t <sub>w</sub>	$1.49 \sqrt{\frac{E}{F_y}}$	
Stiffened Elements	6 <del>Walls of rectangular HSS and boxes of uniform thickness</del>	b/t	$1.40 \sqrt{\frac{E}{F_y}}$	 <del>Deleted: Sides</del> <del>Deleted: and hollow structural sections</del>
	7 Flange cover plates and diaphragm plates between lines of fasteners or welds	b/t	$1.40 \sqrt{\frac{E}{F_y}}$	
	8 All other stiffened elements	b/t	$1.49 \sqrt{\frac{E}{F_y}}$	
	9 Circular <del>HSS</del>	D/t	$0.11 \sqrt{\frac{E}{F_y}}$	 <del>Deleted: hollow sections</del>

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TABLE B4.1b

Table B4.1b					
Limiting Width-Thickness Ratios for Compression Elements in Members Subject to Flexure For Use with Chapter F and Appendix 1					
Case	Description of Element	Width Thickness Ratio	Limiting Width-Thickness Ratios		Example
			$\lambda_p$ (compact / noncompact)	$\lambda_r$ (noncompact / slender)	
Unstiffened Elements	10 Flanges of rolled I-shaped sections, channels, and tees	b/t	$0.38 \sqrt{\frac{E}{F_y}}$	$1.0 \sqrt{\frac{E}{F_y}}$	
	11 Flanges of doubly and singly symmetric I-shaped built-up sections	b/t	$0.38 \sqrt{\frac{E}{F_y}}$	$0.95 \sqrt{\frac{k_c E}{F_y}}$ [a][b]	
	12 Legs of single angles	b/t	$0.54 \sqrt{\frac{E}{F_y}}$	$0.91 \sqrt{\frac{E}{F_y}}$	
	13 Flanges of all I-shaped sections and channels in flexure about the weak axis	b/t	$0.38 \sqrt{\frac{E}{F_y}}$	$1.0 \sqrt{\frac{E}{F_y}}$	
	14 Stems of tees	d/t	$0.84 \sqrt{\frac{E}{F_y}}$	$1.03 \sqrt{\frac{E}{F_y}}$	
Stiffened Elements	15 Webs of doubly-symmetric I-shaped sections and channels	h/t <sub>w</sub>	$3.76 \sqrt{\frac{E}{F_y}}$	$5.70 \sqrt{\frac{E}{F_y}}$	
	16 Webs of singly-symmetric I-shaped sections [c]	h <sub>c</sub> /t <sub>w</sub>	$\frac{h_c}{h_y} \sqrt{\frac{E}{F_y}} \leq \lambda_y$ $\left(0.54 \frac{M_y}{M_x} - 0.09\right)^2$	$5.70 \sqrt{\frac{E}{F_y}}$	
	17 Flanges of rectangular HSS and boxes of uniform thickness	b/t	$1.12 \sqrt{\frac{E}{F_y}}$	$1.40 \sqrt{\frac{E}{F_y}}$	
	18 Flange cover plates and diaphragm plates between lines of fasteners or welds	b/t	$1.12 \sqrt{\frac{E}{F_y}}$	$1.40 \sqrt{\frac{E}{F_y}}$	
	19 Webs of rectangular HSS and boxes of uniform thickness	h/t	$2.42 \sqrt{\frac{E}{F_y}}$	$5.70 \sqrt{\frac{E}{F_y}}$	
	20 Circular HSS	D/t	$0.07 \sqrt{\frac{E}{F_y}}$	$0.31 \sqrt{\frac{E}{F_y}}$	

[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 11)

[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  $S_x/S_{xc} \geq 0.7$ ;  $F_L = F_y S_x/S_{xc} \geq 0.5F_y$  for major-axis bending of compact and noncompact web built-up I-shaped members with  $S_x/S_{xc} < 0.7$ . (See Case 11)

[c]  $M_y$  is the yield moment determined with the minimum elastic section modulus.

## 2. Design Wall Thickness for HSS

The *design wall thickness*,  $t$ , shall be used in calculations involving the wall thickness of hollow structural sections (*HSS*). The design wall thickness,  $t$ , shall be taken equal to 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.

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

ASTM A500 HSS and ASTM A53 Grade B pipe are produced by an ERW process. An SAW process is used for cross sections that are larger than those permitted by ASTM A500.

## 3. Gross and Net Area Determination

### 3a. Gross Area

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

### 3b. Net Area

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

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

For a chain of holes extending across a part in any diagonal or zigzag line, the net width of the part shall be obtained by deducting from the gross width the sum of the diameters or slot dimensions as provided in Section J3.2, 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, in. (mm)

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

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

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



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

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

**User Note:** Section J4.1(b) limits  $A_n$  to a maximum of  $0.85A_g$  for splice plates with holes.

## **B5. FABRICATION, ERECTION AND QUALITY CONTROL**

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

## **B6. EVALUATION OF EXISTING STRUCTURES**

Provisions for the evaluation of existing structures are presented in Appendix 5, Evaluation of Existing Structures.

## CHAPTER C

### DESIGN FOR STABILITY

This chapter addresses requirements for the design of structures for stability. The *direct analysis method* is presented herein; alternative methods are presented in Appendix 7.

The chapter is organized as follows:

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

#### C1. GENERAL STABILITY REQUIREMENTS

Stability shall be provided for the structure as a whole and for each of its elements. The effects of all of the following on the stability of the structure and its elements shall be considered: (1) flexural, shear, and axial member deformations, and all other deformations that contribute to displacements of the structure; (2) *second-order effects* (both  $P-\Delta$  and  $P-\delta$  effects); (3) geometric imperfections; (4) stiffness reductions due to inelasticity; and (5) uncertainty in stiffness and strength. All load-dependent effects shall be calculated at a level of loading corresponding to *LRFD load combinations* or 1.6 times *ASD load combinations*.

Any rational method of design for stability that considers all of the listed effects is permitted.

**User Note:** The term “design” as used in these provisions is the combination of analysis to determine the *required strengths* of components and the proportioning of components to have adequate *available strength*.

The *direct analysis method*, which consists of the calculation of *required strengths* in accordance with Section C2 and the calculation of *available strengths* in accordance with Section C3, is permitted for all structures. The *effective length method* and the *first-order analysis method*, defined in Appendix 7, are permitted as alternatives to the direct analysis method for structures that satisfy the constraints specified in that appendix.

**User Note:** See Commentary Section C1 and Table C-C1.1 for explanation of how requirements (1) through (5) of Section C1 are satisfied in the *direct analysis method*, the *effective length method* and the *first-order analysis method*.

For structures designed by inelastic analysis, the provisions of Appendix 1

shall be satisfied.

## C2. CALCULATION OF REQUIRED STRENGTHS

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

### 1. General Analysis Requirements

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

- (1) The analysis shall be a *second-order analysis* that considers both  $P$ - $\Delta$  and  $P$ - $\delta$  effects, except that it is permissible to neglect the effect of  $P$ - $\delta$  on the response of the structure when it can be shown that the resulting error would be insignificant. Use of the approximate method of second-order analysis provided in Appendix 8 is permitted as an alternative to a rigorous second-order analysis.

**User Note:** It is necessary in all cases to consider the effects of  $P$ - $\delta$  on individual members subject to compression and flexure. However, a  $P$ - $\Delta$ -only second-order analysis (one that neglects the effects of  $P$ - $\delta$  on the response of the structure) would result in little error (typically less than 3% error in lateral displacements) and should be acceptable for typical building structures if the ratio of second-order drift to first-order drift is less than 1.5 and no more than one-third of the total gravity load on the building is on columns that are part of moment-resisting frames in the direction of translation being considered.

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

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

- (4) For design by LRFD, the *second-order analysis* shall be carried out under *LRFD load combinations*. For design by ASD, the *second-order analysis* shall be carried out under 1.6 times the *ASD load combinations*, and the results shall be divided by 1.6 to obtain the

required strengths of components.

## 2. Consideration of Initial Imperfections

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

**User Note:** The imperfections considered in this section are imperfections in the locations of points of intersection of members. In typical building structures, the important imperfection of this type is the out-of-plumbness of columns. Initial out-of-straightness of individual members is not addressed in this section; it is accounted for in the compression member design provisions of Chapter E and need not be considered explicitly in the analysis as long as it is within the limits specified in the *Code of Standard Practice*.

### 2a. Direct Modeling of Imperfections

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

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

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

### 2b. Use of Notional Loads to Represent Imperfections

For structures that support gravity loads primarily through nominally-vertical columns, walls, or frames, it is permissible to use *notional loads* to

represent the effects of initial imperfections in accordance with the requirements of this section. The notional load shall be applied to a model of the structure based on its nominal geometry.

**User Note:** The notional load concept is applicable to all types of structures, but the specific requirements in C2.2b(1) through C2.2b(4) are applicable only for the particular class of structure identified above.

- (1) *Notional loads* shall be applied as *lateral loads* at all levels. The *notional loads* shall be additive to other *lateral loads* and shall be applied in all *load combinations*, except as indicated in (4), below. The magnitude of the *notional loads* shall be:

$$N_i = 0.002Y_i \quad (C2-1)$$

where

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

$Y_i$  = *gravity load* applied at level  $i$  from the *LRFD load combination* or 1.6 times the *ASD load combination*, as applicable, kips (N)

**User Note:** The notional loads can lead to additional (generally small) fictitious base shears in the structure. The correct horizontal reactions at the foundation may be obtained by applying an additional horizontal force at the base of the structure, equal and opposite in direction to the sum of all notional loads, distributed among vertical load-carrying elements in the same proportion as the gravity load supported by those elements.

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

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

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

**User Note:** An out-of-plumbness of 1/500 represents the tolerance on column plumbness specified in the AISC *Code of Standard Practice*.

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

**User Note:** The specified drift ratio threshold of 1.7 is based on analyses using reduced stiffnesses. If the drift ratio is determined from analyses using nominal, unreduced stiffnesses, the equivalent drift ratio is 1.5.

### 3. Adjustments to Stiffness

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

- (1) A factor of 0.8 shall be applied to all stiffnesses that are considered to contribute to the stability of the structure. It is permissible to apply this reduction factor to all stiffnesses in the structure.

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

- (2) An additional factor,  $\tau_b$ , shall be applied to the flexural stiffnesses of all members whose flexural stiffnesses are considered to contribute to the stability of the structure.

- (a) When  $\alpha P_r/P_y \leq 0.5$

$$\tau_b = 1.0 \quad (C3-1a)$$

- (b) When  $\alpha P_r/P_y > 0.5$

$$\tau_b = 4(\alpha P_r/P_y)[1-(\alpha P_r/P_y)] \quad (C3-1b)$$

where

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

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

$P_y$  = axial yield strength ( $= F_y A$ ), kips (N)

**User Note:** Taken together, sections (1) and (2) require the use of  $0.8 EA$  and  $0.8 \tau_b EI$  for structural steel members in the analysis instead of  $EA$  and  $EI$ .

- (3) In structures to which Section C2.2b is applicable, in lieu of using  $\tau_b < 1.0$  where  $\alpha P_r/P_y > 0.5$ , it is permissible to use  $\tau_b = 1.0$  for all members if a *notional load* of  $0.001 Y_i$  (where  $Y_i$  is as defined in Section C2.2b(1)) is applied at all levels, in the direction specified in Section C2.2b(2), in all *load combinations*. These *notional loads* shall be added to those, if any, used to account for imperfections and shall not be subject to Section C2.2b(4).
- (4) Where components comprised of materials other than structural steel are considered to contribute to the stability of the structure and the governing codes and specifications for the other materials require greater reductions in stiffness, such greater stiffness reductions shall be applied to those components.

### C3. CALCULATION OF AVAILABLE STRENGTHS

When *required strengths* have been determined in accordance with Section C2, the *available strengths* of members and connections shall be calculated in accordance with the provisions of Chapters D, E, F, G, H, I, J and K, as applicable, with no further consideration of overall structure stability. The *effective length factor*,  $K$ , of all members shall be taken as unity unless a smaller value can be justified by rational analysis.

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

**User Note:** Methods of satisfying the bracing requirement are provided in Appendix 6. The requirements of Appendix 6 are not applicable to bracing that is included in the analysis of the overall structure as part of the overall load-resisting system.

## CHAPTER D DESIGN OF MEMBERS FOR TENSION

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

The chapter is organized as follows:

- D1. Slenderness Limitations
- D2. Tensile Strength
- D3. Effective Net Area
- D4. Built-Up Members
- D5. Pin-Connected Members
- D6. Eyebars

**User Note:** For cases not included in this chapter the following sections apply:

- B3.10 Members subject to fatigue.
- Chapter H Members subject to combined axial tension and flexure.
- J3. Threaded rods.
- J4.1 Connecting elements in tension.
- J4.3 Block shear rupture strength at end connections of tension members.

### D1. SLENDERNESS LIMITATIONS

There is no maximum slenderness limit for members in tension.

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

### D2. TENSILE STRENGTH

The *design tensile strength*,  $\phi_t P_n$ , and the *allowable tensile strength*,  $P_n/\Omega_t$ , 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:

$$\begin{aligned} P_n &= F_y A_g & (D2-1) \\ \phi_t &= 0.90 \text{ (LRFD)} & \Omega_t = 1.67 \text{ (ASD)} \end{aligned}$$

(b) For tensile rupture in the net section:

$$\begin{aligned} P_n &= F_u A_e & (D2-2) \\ \phi_t &= 0.75 \text{ (LRFD)} & \Omega_t = 2.00 \text{ (ASD)} \end{aligned}$$

where



$A_e$  = effective net area, in.<sup>2</sup> (mm<sup>2</sup>)

$A_g$  = gross area of member, in.<sup>2</sup> (mm<sup>2</sup>)

$F_y$  = specified minimum yield stress of the type of steel being used,  
ksi (MPa)

$F_u$  = specified minimum tensile strength of the type of steel being  
used, ksi (MPa)

When members without holes are fully connected by welds, the effective net area used in Equation D2-2 shall be as defined in Section D3. 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 Equation D2-2.

### D3. EFFECTIVE NET AREA

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

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

$$A_e = A_n U \quad (D3-1)$$


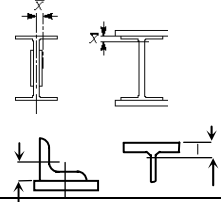

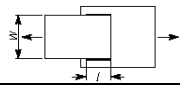
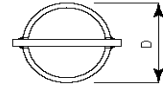
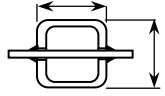
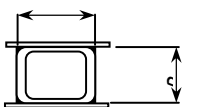




where  $U$ , the shear lag factor, is determined as shown in Table D3.1.

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

**User Note:** For bolted splice plates  $A_e = A_n \leq 0.85A_g$ , according to Section J4).

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**TABLE D3.1**  
**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 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 or by longitudinal welds in combination with transverse welds. (Alternatively, for W, M, S and HP, Case 7 may be used. For angles, Case 8 may be used)		$U = 1 - \bar{x}/l$	
3	All tension members where the tension load is transmitted only by transverse welds to some but not all of the cross-sectional elements.		$U = 1.0$ and $A_n$ = area of the directly connected elements	
4	Plates where the tension load is transmitted by longitudinal welds only.		$l \geq 2w \dots U = 1.0$ $2w > l \geq 1.5w \dots U = 0.87$ $1.5w > l \geq w \dots U = 0.75$	
5	Round HSS with a single concentric gusset plate		$l \geq 1.3D \dots U = 1.0$ $D \leq 1.3D \dots U = 1 - \bar{x}/l$ $\bar{x} = D/\pi$	
6	Rectangular HSS	with a single concentric gusset plate	$l \geq H \dots U = 1 - \bar{x}/l$ $\bar{x} = \frac{B^2 + 2BH}{4(B + H)}$	
		with two side gusset plates	$l \geq H \dots U = 1 - \bar{x}/l$ $\bar{x} = \frac{B^2}{4(B + H)}$	
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	$b_f \geq 2/3 d \dots U = 0.90$ $b_f < 2/3 d \dots U = 0.85$	
		with web connected with 4 or more fasteners per line in the direction of loading	$U = 0.70$	
8	Single and double angles (If $U$ is calculated per Case 2, the larger value is permitted to be used)	with 4 or more fasteners per line in direction of loading	$U = 0.80$	
		with 3 fasteners per line in the direction of loading (with fewer than 3 fasteners per line in the direction of loading, use Case 2)	$U = 0.60$	

$l$  = length of connection, in. (mm);  $w$  = plate width, in. (mm);  $\bar{x}$  = connection eccentricity, in. (mm);  $B$  = overall width of rectangular HSS member, measured 90° to the plane of the connection, in. (mm);  $H$  = overall height of rectangular HSS member, measured in the plane of the connection, in. (mm)

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#### D4. BUILT-UP MEMBERS

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

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 6 in. (150 mm).

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

#### D5. PIN-CONNECTED MEMBERS

##### 1. Tensile Strength

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

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

$$P_n = 2tb_e F_u \quad (D5-1)$$

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

(b) For shear rupture on the effective area:

$$P_n = 0.6F_u A_{sf} \quad (D5-2)$$

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

where

- $A_{sf}$  = area on the shear failure path =  $2t(a + d/2)$ , in.<sup>2</sup> (mm<sup>2</sup>)
- $a$  = shortest distance from edge of the pin hole to the edge of the member measured parallel to the direction of the *force*, in. (mm)
- $b_e$  =  $2t + 0.63$ , in. ( $= 2t + 16$ , mm), but not more than the actual distance from the edge of the hole to the edge of the part measured in the direction normal to the applied force, in. (mm)

$d$  = pin diameter, in. (mm)  
 $t$  = thickness of plate, in. (mm)

(c) For bearing on the projected area of the pin, see Section J7.

(d) For yielding on the gross section, use Section D2(a).

## 2. Dimensional Requirements

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  $\frac{1}{32}$  in. (1 mm) greater than the diameter of the pin.

The width of the plate at the pin hole shall not be less than  $2b_e + d$  and the minimum extension,  $a$ , beyond the bearing end of the pin hole, parallel to the axis of the member, shall not be less than  $1.33 \times b_e$ .

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

## D6. EYEBARS

### 1. Tensile Strength

The *available tensile strength* of *eyebars* shall be determined in accordance with Section D2, with  $A_g$  taken as the cross-sectional area of the body.

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

### 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  $\frac{1}{32}$  in. (1 mm) greater than the pin diameter.

For steels having  $F_y$  greater than 70 ksi (485 MPa), the hole diameter shall not exceed five times the plate thickness, and the width of the

856 eyebar body shall be reduced accordingly.

857  
858 A thickness of less than  $\frac{1}{2}$  in. (13 mm) is permissible only if external  
859 nuts are provided to tighten pin plates and *filler* plates into snug  
860 contact. The width from the hole edge to the plate edge perpendicular  
861 to the direction of applied *load* shall be greater than two-thirds and, for  
862 the purpose of calculation, not more than three-fourths times the  
863 eyebar body width.  
864

## CHAPTER E

### DESIGN OF MEMBERS FOR COMPRESSION

This chapter addresses members subject to axial compression through the centroidal axis. The chapter is organized as follows:

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

**User Note:** For cases not included in this chapter the following sections apply:

- H1. – H3. Members subject to combined axial compression and flexure.
- H4. Members subject to axial compression and torsion.
- J4.4 Compressive strength of connecting elements.
- I2. Composite axially loaded members.







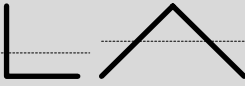

#### E1. GENERAL PROVISIONS

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

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

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

**Table User Note E1.1**  
**Selection Table for the Application of Chapter E Sections**

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

## E2. EFFECTIVE LENGTH

The effective length factor,  $K$ , for calculation of column slenderness,  $KL/r$ , shall be determined in accordance with Chapter C or Appendix 7,

where

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

$r$  = radius of gyration, in. (mm)

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

### E3. FLEXURAL BUCKLING OF MEMBERS WITHOUT SLENDER ELEMENTS

This section applies to compression members without slender elements, as defined in Section B4.1 for elements in uniform compression.

**User Note:** When the torsional unbraced length is larger than the lateral unbraced length, Section E4 may control the design of wide flange and similarly shaped columns.

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

$$P_n = F_{cr} A_g \quad (E3-1)$$

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

$$(a) \text{ When } \frac{KL}{r} \leq 4.71 \sqrt{\frac{E}{F_y}} \quad \left( \text{or } \frac{F_y}{F_e} \leq 2.25 \right)$$

$$F_{cr} = \left[ 0.658 \frac{F_y}{F_e} \right] F_y \quad (E3-2)$$

$$(b) \text{ When } \frac{KL}{r} > 4.71 \sqrt{\frac{E}{F_y}} \quad \left( \text{or } \frac{F_y}{F_e} > 2.25 \right)$$

$$F_{cr} = 0.877 F_e \quad (E3-3)$$

where

$F_e$  = elastic buckling stress determined according to Equation E3-4, as specified in Appendix Section 7.2.3(b), or through an elastic buckling analysis, as applicable, ksi (MPa)

$$F_e = \frac{\pi^2 E}{\left( \frac{KL}{r} \right)^2} \quad (E3-4)$$

**User Note:** The two equations for calculating the limits and applicability of Sections E3(a) and E3(b), one based on  $KL/r$  and one based on  $F_y/F_e$ , provide the



same result.

#### E4. 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* without slender elements, as defined in Section B4.1 for elements in uniform compression. In addition, this section applies to all doubly symmetric members without slender elements when the torsional unbraced length exceeds the lateral unbraced length. These provisions are not required for single angles with  $b/t \leq 20$ , which are covered in Section E5.

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

$$P_n = F_{cr} A_g \quad (E4-1)$$

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

(a) For double angle and tee-shaped compression members:

$$F_{cr} = \left( \frac{F_{cry} + F_{crz}}{2H} \right) \left[ 1 - \sqrt{1 - \frac{4F_{cry}F_{crz}H}{(F_{cry} + F_{crz})^2}} \right] \quad (E4-2)$$

where  $F_{cry}$  is taken as  $F_{cr}$  from Equation E3-2 or E3-3, for flexural buckling about the  $y$ -axis of symmetry and  $\frac{KL}{r} = \frac{K_y L}{r_y}$  for T-shaped compression

members and  $\frac{KL}{r} = \left( \frac{KL}{r} \right)_m$  from Section E6 for double angle compression members, and

$$F_{crz} = \frac{GJ}{A_g \bar{r}_o^2} \quad (E4-3)$$

(b) For all other cases,  $F_{cr}$  shall be determined according to Equation E3-2 or E3-3, using the torsional or flexural-torsional elastic buckling stress,  $F_e$ , determined as follows:

(i) For doubly symmetric members:

$$F_e = \left[ \frac{\pi^2 E C_w}{(K_z L)^2} + GJ \right] \frac{1}{I_x + I_y} \quad (E4-4)$$

(ii) For singly symmetric members where  $y$  is the axis of symmetry:

$$F_e = \left( \frac{F_{ey} + F_{ez}}{2H} \right) \left[ 1 - \sqrt{1 - \frac{4F_{ey}F_{ez}H}{(F_{ey} + F_{ez})^2}} \right] \quad (E4-5)$$

(iii) For unsymmetric members,  $F_e$  is the lowest root of the cubic equation:

$$(F_e - F_{ex})(F_e - F_{ey})(F_e - F_{ez}) - F_e^2(F_e - F_{ey})\left(\frac{x_o}{\bar{r}_o}\right)^2 - F_e^2(F_e - F_{ex})\left(\frac{y_o}{\bar{r}_o}\right)^2 = 0 \quad (\text{E4-6})$$

where

$A_g$  = gross area of member, in.<sup>2</sup> (mm<sup>2</sup>)  
 $C_w$  = warping constant, in.<sup>6</sup> (mm<sup>6</sup>)

$$F_{ex} = \frac{\pi^2 E}{\left(\frac{K_x L}{r_x}\right)^2} \quad (\text{E4-7})$$

$$F_{ey} = \frac{\pi^2 E}{\left(\frac{K_y L}{r_y}\right)^2} \quad (\text{E4-8})$$

$$F_{ez} = \frac{\left(\frac{\pi^2 E C_w}{(K_z L)^2} + GJ\right)}{A_g \bar{r}_o^2} \quad (\text{E4-9})$$

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

$$H = 1 - \frac{x_o^2 + y_o^2}{\bar{r}_o^2} \quad (\text{E4-10})$$

$I_x, I_y$  = moment of inertia about the principal axes, in.<sup>4</sup> (mm<sup>4</sup>)

$J$  = torsional constant, in.<sup>4</sup> (mm<sup>4</sup>)

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

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

$K_z$  = effective length factor for torsional buckling

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

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

$$\bar{r}_o^2 = x_o^2 + y_o^2 + \frac{I_x + I_y}{A_g} \quad (\text{E4-11})$$

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

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

**User Note:** For doubly symmetric I-shaped sections  $C_w$  may be taken as  $I_x h_o^2/4$ , where  $h_o$  is the distance between flange centroids, in lieu of a more precise analysis. For tees and double angles, omit term with  $C_w$  when computing  $F_{ez}$  and take  $x_o$  as 0.

## E5. SINGLE ANGLE COMPRESSION MEMBERS

The *nominal compressive strength*,  $P_n$ , of single angle members shall be determined in accordance with Section E3 or Section E7, as appropriate, for axially loaded members. Members meeting the criteria imposed in Section E5(a) or E5(b) are permitted to be designed as axially loaded members using the specified effective slenderness ratio,  $KL/r$ .

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

- (1) members are loaded at the ends in compression through the same one leg;
- (2) members are attached by welding or by *connections* with a minimum of two bolts; and
- (3) there are no intermediate transverse *loads*.

Single angle members with different end conditions from those described in Section E5(a) or (b), with the ratio of long leg width to short leg width greater than 1.7 or with transverse loading shall be evaluated for combined axial *load* and flexure using the provisions of Chapter H.

(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  $\frac{L}{r_x} \leq 80$ :

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

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

$$\frac{KL}{r} = 32 + 1.25 \frac{L}{r_x} \leq 200 \quad (\text{E5-2})$$

For unequal-leg angles with leg length ratios less than 1.7 and connected through the shorter leg,  $KL/r$  from Equations E5-1 and E5-2 shall be increased by adding  $4[(b_l/b_s)^2 - 1]$ , but  $KL/r$  of the members shall not be taken as 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  $\frac{L}{r_x} \leq 75$ :

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

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

$$\frac{KL}{r} = 45 + \frac{L}{r_x} \leq 200 \quad (\text{E5-4})$$

For unequal-leg angles with leg length ratios less than 1.7 and connected

through the shorter leg,  $KL/r$  from Equations E5-3 and E5-4 shall be increased by adding  $6[(b/b_s)^2 - 1]$ , but  $KL/r$  of the member shall not be taken as less than  $0.82L/r_z$ ,

where

$L$  = length of member between work points at truss chord centerlines, in. (mm)

$b_l$  = length of longer leg of angle, in. (mm)

$b_s$  = length of shorter leg of angle, in. (mm)

$r_x$  = radius of gyration about *geometric axis* parallel to connected leg, in. (mm)

$r_z$  = radius of gyration about the minor principal axis, in. (mm)

## E6. BUILT-UP MEMBERS

### 1. Compressive Strength

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

**User Note:** It is acceptable to design a bolted end *connection* of a built-up compression member with the bolts in shear as a non-slip-critical connection; however, the bolts must be pretensioned. The requirement for Class A or B faying surfaces is not intended for the resistance of the axial force in the built-up member, but rather to prevent relative movement between the components at the end as the built-up member takes a curved shape.

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

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

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

(b) For intermediate connectors that are welded or are connected by means of pretensioned bolts:

(i) When  $\left(\frac{a}{r_i}\right) \leq 40$

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

(ii) When  $\left(\frac{a}{r_i}\right) > 40$

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

where

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

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

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

$a$  = distance between connectors, in. (mm)

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

## 2. Dimensional Requirements

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  $Ka / r_i$  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.

At the ends of built-up compression members bearing on base plates or finished surfaces, all components in contact with one another shall be connected by a weld having a length not less than the maximum width of the member or by bolts spaced longitudinally not more than four diameters apart for a distance equal to 1½ 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 strength*. For limitations on the longitudinal spacing of fasteners between elements in continuous contact consisting of a plate and a shape or two plates, see Section J3.5. Where a component of a built-up compression member consists of an outside plate, the maximum spacing shall not exceed the thickness of the thinner outside plate times  $0.75\sqrt{E / F_y}$ , nor 12 in. (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 of fasteners on each *gage* line shall not exceed the thickness of the thinner outside plate times  $1.12\sqrt{E / F_y}$  nor 18 in. (460 mm).

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 Section B4.1, 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 Section B4.1.

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

(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 1½ in. (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 element included between their connections shall not exceed three-fourths times the governing slenderness ratio for the member as a whole. Lacing shall be proportioned to provide a shearing strength normal to the axis of the member equal to 2 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% of that distance for double lacing.

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

For additional spacing requirements, see Section J3.5.

## E7. MEMBERS WITH SLENDER ELEMENTS

This section applies to compression members with slender elements, as defined in Section B4.1 for elements in uniform compression.

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

$$P_n = F_{cr} A_g \quad (E7-1)$$

The critical stress,  $F_{cr}$ , shall be determined as follows:

$$(a) \text{ When } \frac{KL}{r} \leq 4.71 \sqrt{\frac{E}{QF_y}} \quad \left( \text{or } \frac{QF_y}{F_e} \leq 2.25 \right)$$

$$F_{cr} = Q \left[ 0.658 \frac{QF_y}{F_e} \right] F_y \quad (E7-2)$$

$$(b) \text{ When } \frac{KL}{r} > 4.71 \sqrt{\frac{E}{QF_y}} \quad \left( \text{or } \frac{QF_y}{F_e} > 2.25 \right)$$

$$F_{cr} = 0.877 F_e \quad (E7-3)$$

where

$F_e$  = elastic buckling stress, calculated using Equations E3-4 and E4-4 for doubly symmetric members, Equations E3-4 and E4-5 for singly symmetric members, and Equation E4-6 for unsymmetric members, except for single angles where  $F_e$  is calculated using Equation E3-4.

$Q$  = net reduction factor accounting for all slender compression elements; = 1.0 for members without slender elements, as defined in Section B4.1, for elements in uniform compression

=  $Q_s Q_a$  for members with *slender-element sections*, as defined in Section B4.1, for elements in uniform compression.

**User Note:** For cross sections composed of only unstiffened slender elements,  $Q = Q_s$  ( $Q_a = 1.0$ ). For cross sections composed of only stiffened slender elements,  $Q = Q_a$  ( $Q_s = 1.0$ ). For cross sections composed of both stiffened and unstiffened slender elements,  $Q = Q_s Q_a$ . For cross sections composed of multiple unstiffened slender elements, it is conservative to use the smaller  $Q_s$  from the more slender element in determining the member strength for pure compression.

### 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:

(i) When  $\frac{b}{t} \leq 0.56 \sqrt{\frac{E}{F_y}}$

$$Q_s = 1.0 \quad (\text{E7-4})$$

(ii) When  $0.56 \sqrt{E/F_y} < b/t < 1.03 \sqrt{E/F_y}$

$$Q_s = 1.415 - 0.74 \left( \frac{b}{t} \right) \sqrt{\frac{F_y}{E}} \quad (\text{E7-5})$$

(iii) When  $b/t \geq 1.03 \sqrt{E/F_y}$

$$Q_s = \frac{0.69E}{F_y \left( \frac{b}{t} \right)^2} \quad (\text{E7-6})$$

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

(i) When  $\frac{b}{t} \leq 0.64 \sqrt{\frac{Ek_c}{F_y}}$

$$Q_s = 1.0 \quad (\text{E7-7})$$

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

$$Q_s = 1.415 - 0.65 \left( \frac{b}{t} \right) \sqrt{\frac{F_y}{Ek_c}} \quad (\text{E7-8})$$

(iii) When  $b/t > 1.17 \sqrt{\frac{Ek_c}{F_y}}$

$$Q_s = \frac{0.90Ek_c}{F_y \left( \frac{b}{t} \right)^2} \quad (\text{E7-9})$$

where

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

- (c) For single angles



$$(i) \text{ When } \frac{b}{t} \leq 0.45 \sqrt{\frac{E}{F_y}} \quad Q_s = 1.0 \quad (E7-10)$$

$$(ii) \text{ When } 0.45 \sqrt{\frac{E}{F_y}} < b/t \leq 0.91 \sqrt{\frac{E}{F_y}} \quad Q_s = 1.34 - 0.76 \left( \frac{b}{t} \right) \sqrt{\frac{F_y}{E}} \quad (E7-11)$$

$$(iii) \text{ When } b/t > 0.91 \sqrt{\frac{E}{F_y}} \quad Q_s = \frac{0.53E}{F_y \left( \frac{b}{t} \right)^2} \quad (E7-12)$$

where

$b$  = full width of longest leg, in. (mm)

(d) For stems of tees

$$(i) \text{ When } \frac{d}{t} \leq 0.75 \sqrt{\frac{E}{F_y}} \quad Q_s = 1.0 \quad (E7-13)$$

$$(ii) \text{ When } 0.75 \sqrt{\frac{E}{F_y}} < d/t \leq 1.03 \sqrt{\frac{E}{F_y}} \quad Q_s = 1.908 - 1.22 \left( \frac{d}{t} \right) \sqrt{\frac{F_y}{E}} \quad (E7-14)$$

$$(iii) \text{ When } d/t > 1.03 \sqrt{\frac{E}{F_y}} \quad Q_s = \frac{0.69E}{F_y \left( \frac{d}{t} \right)^2} \quad (E7-15)$$

where

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

$d$  = full depth of tee, in. (mm)

$t$  = thickness of element, in. (mm)

## 2. Slender Stiffened Elements, $Q_a$

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

$$Q_a = \frac{A_e}{A} \quad (\text{E7-16})$$

where

$A$  = total cross-sectional area of member, in.<sup>2</sup> (mm<sup>2</sup>)

$A_e$  = summation of the effective areas of the cross section based on the reduced *effective width*,  $b_e$ , in.<sup>2</sup> (mm<sup>2</sup>)

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:

$$b_e = 1.92t \sqrt{\frac{E}{f}} \left[ 1 - \frac{0.34}{(b/t)} \sqrt{\frac{E}{f}} \right] \leq b \quad (\text{E7-17})$$

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

$$b_e = 1.92t \sqrt{\frac{E}{f}} \left[ 1 - \frac{0.38}{(b/t)} \sqrt{\frac{E}{f}} \right] \leq b \quad (\text{E7-18})$$

where

$f = P_n / A_e$

**User Note:** In lieu of calculating  $f = P_n / A_e$ , which requires iteration,  $f$  may be taken equal to  $F_y$ . This will result in a slightly conservative estimate of column capacity.

- (c) For axially loaded circular sections:

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

$$Q = Q_a = \frac{0.038E}{F_y(D/t)} + \frac{2}{3} \quad (\text{E7-19})$$

where

$D$  = outside diameter, in. (mm)

$t$  = thickness of wall, in. (mm)

## CHAPTER F

### DESIGN OF MEMBERS FOR FLEXURE

This chapter applies to members subject to simple bending about one principal axis. For simple bending, the member is loaded in a plane parallel to a principal axis that passes through the shear center or is restrained against twisting at *load* points and supports.

The chapter is organized as follows:

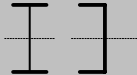

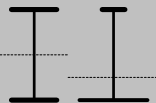
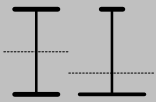
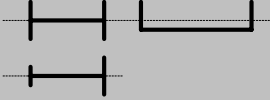

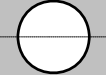

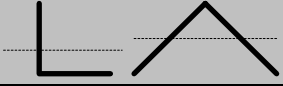

- F1. General Provisions
- F2. Doubly Symmetric Compact I-Shaped Members and Channels Bent about Their Major Axis
- F3. Doubly Symmetric I-Shaped Members with Compact Webs and Noncompact or Slender Flanges Bent about Their Major Axis
- F4. Other I-Shaped Members with Compact or Noncompact Webs Bent about Their Major Axis
- F5. Doubly Symmetric and Singly Symmetric I-Shaped Members with Slender Webs Bent about Their Major Axis
- F6. I-Shaped Members and Channels Bent about Their Minor Axis
- F7. Square and Rectangular HSS and Box-Shaped Members
- F8. Round HSS
- F9. Tees and Double Angles Loaded in the Plane of Symmetry
- F10. Single Angles
- F11. Rectangular Bars and Rounds
- F12. Unsymmetrical Shapes
- F13. Proportions of Beams and Girders

**User Note:** For cases not included in this chapter the following sections apply:

- H1–H3. Members subject to biaxial flexure or to combined flexure and axial force.
- H4. Members subject to flexure and torsion.
- Appendix 3. Members subject to fatigue.
- Chapter G. Design provisions for shear.

Table User Note F1.1

**Selection Table for the Application of Chapter F Sections**

Section in Chapter F	Cross Section	Flange Slender-ness	Web Slender-ness	Limit States
F2		C	C	Y, LTB
F3		NC, S	C	LTB, FLB
F4		C, NC, S	C, NC	Y,LTB, FLB, TFY
F5		C, NC, S	S	Y,LTB, FLB, TFY
F6		C, NC, S	N/A	Y FLB
F7		C, NC, S	C, NC	Y, FLB, WLB
F8		N/A	N/A	Y, LB
F9		C, NC, S	N/A	Y, LTB, FLB
F10		N/A	N/A	Y, LTB, LLB
F11		N/A	N/A	Y, LTB
F12	Unsymmetrical shapes, other than single angles	N/A	N/A	All limit states

Y = yielding, 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

**F1. GENERAL PROVISIONS**

FOR COMMITTEE USE ONLY

Specification for Structural Steel Buildings, Public Review Draft dated March 1, 2009

AMERICAN INSTITUTE OF STEEL CONSTRUCTION

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

- (1) For all provisions in this chapter

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

and the *nominal flexural strength*,  $M_n$ , shall be determined according to Sections F2 through F13.

- (2) 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.

- (3) For singly symmetric members in single curvature and all doubly symmetric members:

$C_b$  = *lateral-torsional buckling* modification factor for nonuniform moment diagrams when both ends of the segment are braced

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

where

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

$M_A$  = absolute value of moment at quarter point of the unbraced segment, kip-in. (N-mm)

$M_B$  = absolute value of moment at centerline of the unbraced segment, kip-in. (N-mm)

$M_C$  = absolute value of moment at three-quarter point of the unbraced segment, kip-in. (N-mm)

For cantilevers or overhangs where the free end is unbraced,  $C_b = 1.0$ .

**User Note:** For doubly symmetric members with no transverse loading between brace points, Equation F1-1 reduces to 2.27 for the case of equal end moments of opposite sign and to 1.67 when one end moment equals zero. For singly symmetric members, a more detailed analysis for  $C_b$  is presented in the Commentary.

- (4) In singly symmetric members subject to *reverse curvature* bending, the *lateral-torsional buckling* strength shall be checked for both flanges. The available flexural strength shall be greater than or equal to the maximum required moment causing compression within the flange under consideration.

## F2. DOUBLY SYMMETRIC COMPACT I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MAJOR AXIS

This section applies to doubly symmetric I-shaped members and channels bent about their major axis, having compact webs and compact flanges as defined in Section B4.1 for flexure.

**User Note:** All current ASTM A6 W, S, M, C and MC shapes except W21x48, W14x99, W14x90, W12x65, W10X12, W8x31, W8x10, W6x15, W6x9, W6x8.5 and M4x6 have compact flanges for  $F_y = 50$  ksi (345 MPa); all current ASTM A6 W, S, M, HP, C and MC shapes have compact webs at  $F_y \leq 65$  ksi (450 MPa).

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

### 1. Yielding

$$M_n = M_p = F_y Z_x \quad (\text{F2-1})$$

where

$F_y$  = specified minimum yield stress of the type of steel being used, ksi (MPa)

$Z_x$  = plastic section modulus about the x-axis, in.<sup>3</sup> (mm<sup>3</sup>)

### 2. Lateral-Torsional Buckling

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

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

$$M_n = C_b \left[ M_p - (M_p - 0.7 F_y S_x) \left( \frac{L_b - L_p}{L_r - L_p} \right) \right] \leq M_p \quad (\text{F2-2})$$

(c) When  $L_b > L_r$

$$M_n = F_{cr} S_x \leq M_p \quad (\text{F2-3})$$

where

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

$$F_{cr} = \frac{C_b \pi^2 E}{\left( \frac{L_b}{r_{ts}} \right)^2} \sqrt{1 + 0.078 \frac{J C}{S_x h_o \left( \frac{L_b}{r_{ts}} \right)^2}} \quad (\text{F2-4})$$

and where

$E$  = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)

$J$  = torsional constant, in.<sup>4</sup> (mm<sup>4</sup>)

$S_x$  = elastic section modulus taken about the x-axis, in.<sup>3</sup> (mm<sup>3</sup>)  
 $h_o$  = distance between the flange centroids, in. (mm)

**User Note:** The square root term in Equation F2-4 may be conservatively taken equal to 1.0.

The limiting lengths  $L_p$  and  $L_r$  are determined as follows:

$$L_p = 1.76r_y \sqrt{\frac{E}{F_y}} \quad (\text{F2-5})$$

$$L_r = 1.95r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{S_x h_o} + \sqrt{\left(\frac{Jc}{S_x h_o}\right)^2 + 6.76 \left(\frac{0.7F_y}{E}\right)^2}} \quad (\text{F2-6})$$

where

$$r_{ts}^2 = \frac{\sqrt{I_y C_w}}{S_x} \quad (\text{F2-7})$$

and the coefficient  $c$  is determined as follows:

(a) For doubly symmetric I-shapes:  $c = 1$  (F2-8a)

(b) For channels:  $c = \frac{h_o}{2} \sqrt{\frac{I_y}{C_w}}$  (F2-8b)

**User Note:**

For doubly symmetric I-shapes with rectangular flanges,

$C_w = \frac{I_y h_o^2}{4}$  and thus Equation F2-7 becomes

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

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

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

### F3. DOUBLY SYMMETRIC I-SHAPED MEMBERS WITH COMPACT WEBS AND NONCOMPACT OR SLENDER FLANGES BENT ABOUT THEIR MAJOR AXIS

This section applies to doubly symmetric I-shaped members bent about their major axis having compact webs and noncompact or slender flanges as defined in Section B4.1 for flexure.

**User Note:** The following shapes have noncompact flanges for

$F_y = 50$  ksi (345 MPa): W21x48, W14x99, W14x90, W12x65, W10x12, W8x31, W8x10, W6x15, W6x9, W6x8.5 and M4x6. All other ASTM A6 W, S, M, and HP shapes have compact flanges for  $F_y \leq 50$  ksi (345 MPa).

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

#### 1. Lateral-Torsional Buckling

For *lateral-torsional buckling*, the provisions of Section F2.2 shall apply.

#### 2. Compression Flange Local Buckling

(a) For sections with noncompact flanges

$$M_n = M_p - (M_p - 0.7F_y S_x) \left( \frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \quad (\text{F3-1})$$

(b) For sections with slender flanges

$$M_n = \frac{0.9Ek_c S_x}{\lambda^2} \quad (\text{F3-2})$$

where

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

$\lambda_{pf} = \lambda_p$  is the limiting slenderness for a compact flange, Table B4.1

$\lambda_{rf} = \lambda_r$  is the limiting slenderness for a noncompact flange, Table B4.1

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

$h$  = distance as defined in Section B4.1b, in.

#### F4. 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 B4.1 for flexure.

**User Note:** I-shaped members for which this section is applicable may be designed conservatively using Section F5.



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

## 1. Compression Flange Yielding

$$M_n = R_{pc}M_{yc} = R_{pc}F_yS_{xc} \quad (F4-1)$$

## 2. Lateral-Torsional Buckling

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

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

$$M_n = C_b \left[ R_{pc}M_{yc} - (R_{pc}M_{yc} - F_yS_{xc}) \left( \frac{L_b - L_p}{L_r - L_p} \right) \right] \leq R_{pc}M_{yc} \quad (F4-2)$$

(c) When  $L_b > L_r$

$$M_n = F_{cr}S_{xc} \leq R_{pc}M_{yc} \quad (F4-3)$$

where

$M_{yc}$  = yield moment in the compression flange

$$M_{yc} = F_yS_{xc} \quad (F4-4)$$

$$F_{cr} = \frac{C_b \pi^2 E}{\left( \frac{L_b}{r_t} \right)^2} \sqrt{1 + 0.078 \frac{J}{S_{xc} h_o} \left( \frac{L_b}{r_t} \right)^2} \quad (F4-5)$$

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

where

$I_{yc}$  = moment of inertia of the compression flange about y-axis, in.<sup>4</sup>

The stress,  $F_L$ , is determined as follows:

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

$$F_L = 0.7F_y \quad (F4-6a)$$

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

$$F_L = F_y \frac{S_{xt}}{S_{xc}} \geq 0.5F_y \quad (F4-6b)$$

The limiting laterally unbraced length for the limit state of *yielding*,  $L_p$ , is determined as:

$$L_p = 1.1r_t \sqrt{\frac{E}{F_y}} \quad (\text{F4-7})$$

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

$$L_r = 1.95r_t \frac{E}{F_L} \sqrt{\frac{J}{S_{xc}h_o} + \sqrt{\left(\frac{J}{S_{xc}h_o}\right)^2 + 6.76\left(\frac{F_L}{E}\right)^2}} \quad (\text{F4-8})$$

The web *plastification* factor,  $R_{pc}$ , shall be determined as follows:

When  $I_{yc}/I_y > 0.23$

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

$$R_{pc} = \frac{M_p}{M_{yc}} \quad (\text{F4-9a})$$

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

$$R_{pc} = \left[ \frac{M_p}{M_{yc}} - \left( \frac{M_p}{M_{yc}} - 1 \right) \left( \frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}} \right) \right] \leq \frac{M_p}{M_{yc}} \quad (\text{F4-9b})$$

When  $I_{yc}/I_y \leq 0.23$

$$R_{pc} = 1.0$$

where

$M_p$  =  $Z_x F_y \leq 1.6 S_{xc} F_y$   
 $S_{xc}$ ,  $S_{xt}$  = elastic section modulus referred to compression and tension flanges, respectively, in.<sup>3</sup> (mm<sup>3</sup>)

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

$\lambda_{pw}$  =  $\lambda_p$ , the limiting slenderness for a compact web, Table B4.1

$\lambda_{rw}$  =  $\lambda_r$ , the limiting slenderness for a noncompact web, Table B4.1

$h_c$  = 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, in. (mm)

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

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

$$r_t = \frac{b_{fc}}{\sqrt{12 \left( \frac{h_o}{d} + \frac{1}{6} a_w \frac{h^2}{h_o d} \right)}} \quad (\text{F4-10})$$

where

$$a_w = \frac{h_c t_w}{b_{fc} t_{fc}} \quad (\text{F4-11})$$

$b_{fc}$  = compression flange width, in. (mm)

$t_{fc}$  = compression flange thickness, in. (mm)

(ii) For I-shapes with a channel cap or a *cover plate* attached to the compression flange

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

$a_w$  = 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

**User Note:** For I-shapes with a rectangular compression flange,  $r_t$  may be approximated accurately and conservatively as the radius of gyration of the compression flange plus one-third of the compression portion of the web; in other words,

$$r_t = \frac{b_{fc}}{\sqrt{12 \left( 1 + \frac{1}{6} a_w \right)}} .$$

### 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

$$M_n = \left[ R_{pc} M_{yc} - (R_{pc} M_{yc} - F_L S_{xc}) \left( \frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right] \quad (\text{F4-12})$$

(c) For sections with slender flanges

$$M_n = \frac{0.9 E k_c S_{xc}}{\lambda^2} \quad (\text{F4-13})$$

where

$F_L$  is defined in Equations F4-6a and F4-6b

$R_{pc}$  is the web *plastification* factor, determined by Equations F4-9

1700  $k_c = \frac{4}{\sqrt{h/t_w}}$  and shall not be taken less than 0.35 nor greater than  
 1701 0.76 for calculation purposes

1702  $\lambda = \frac{b_{fc}}{2t_{fc}}$

1703  $\lambda_{pf} = \lambda_p$ , the limiting slenderness for a compact flange, Table B4.1

1704  $\lambda_{rf} = \lambda_r$ , the limiting slenderness for a noncompact flange, Table  
 1705 B4.1

#### 1707 4. Tension Flange Yielding

1709 (a) When  $S_{xt} \geq S_{xc}$ , the *limit state* of tension flange *yielding* does not  
 1710 apply.

1711 (b) When  $S_{xt} < S_{xc}$

$$1712 M_n = R_{pt} M_{yt} \quad (F4-14)$$

1713 where

$$1714 M_{yt} = F_y S_{xt}$$

1715 The web *plastification* factor corresponding to the tension flange  
 1716 yielding limit state,  $R_{pt}$ , is determined as follows:

1719 (i) When  $\frac{h_c}{t_w} \leq \lambda_{pw}$

$$1720 R_{pt} = \frac{M_p}{M_{yt}} \quad (F4-15a)$$

1721 (ii) When  $\frac{h_c}{t_w} > \lambda_{pw}$

$$1722 R_{pt} = \left[ \frac{M_p}{M_{yt}} - \left( \frac{M_p}{M_{yt}} - 1 \right) \left( \frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}} \right) \right] \leq \frac{M_p}{M_{yt}} \quad (F4-15b)$$

1723 where

$$1724 \lambda = \frac{h_c}{t_w}$$

1725  $\lambda_{pw} = \lambda_p$ , the limiting slenderness for a compact web,  
 1726 defined in Table B4.1

1727  $\lambda_{rw} = \lambda_r$ , the limiting slenderness for a noncompact web,  
 1728 defined in Table B4.1

#### 1730 F5. DOUBLY SYMMETRIC AND SINGLY SYMMETRIC I- 1731 SHAPED MEMBERS WITH SLENDER WEBS BENT ABOUT 1732 THEIR MAJOR AXIS

1734 This section applies to doubly symmetric and singly symmetric I-  
 1735 shaped members with slender webs attached to the mid-width of the  
 1736 flanges and bent about their major axis as defined in Section B4.1 for  
 1737 flexure.

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

## 1. Compression Flange Yielding

$$M_n = R_{pg} F_y S_{xc} \quad (\text{F5-1})$$

## 2. Lateral-Torsional Buckling

$$M_n = R_{pg} F_{cr} S_{xc} \quad (\text{F5-2})$$

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

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

$$F_{cr} = C_b \left[ F_y - (0.3 F_y) \left( \frac{L_b - L_p}{L_r - L_p} \right) \right] \leq F_y \quad (\text{F5-3})$$

(c) When  $L_b > L_r$

$$F_{cr} = \frac{C_b \pi^2 E}{\left( \frac{L_b}{r_t} \right)^2} \leq F_y \quad (\text{F5-4})$$

where

$L_p$  is defined by Equation F4-7

$$L_r = \pi r_t \sqrt{\frac{E}{0.7 F_y}} \quad (\text{F5-5})$$

$R_{pg}$  is the bending strength reduction factor:

$$R_{pg} = 1 - \frac{a_w}{1200 + 300 a_w} \left( \frac{h_c}{t_w} - 5.7 \sqrt{\frac{E}{F_y}} \right) \leq 1.0 \quad (\text{F5-6})$$

$a_w$  is defined by Equation F4-11 but shall not exceed 10.

$r_t$  is the effective radius of gyration for lateral buckling as defined in Section F4.

### 3. Compression Flange Local Buckling

$$M_n = R_{pg} F_{cr} S_{xc} \quad (\text{F5-7})$$

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

(b) For sections with noncompact flanges

$$F_{cr} = \left[ F_y - (0.3 F_y) \left( \frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right] \quad (\text{F5-8})$$

(c) For sections with slender flanges

$$F_{cr} = \frac{0.9 E k_c}{\left( \frac{b_f}{2 t_f} \right)^2} \quad (\text{F5-9})$$

where

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

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

$\lambda_{pf} = \lambda_p$ , the limiting slenderness for a compact flange, Table B4.1

$\lambda_{rf} = \lambda_r$ , the limiting slenderness for a noncompact flange, Table B4.1

### 4. Tension Flange Yielding

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

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

$$M_n = F_y S_{xt} \quad (\text{F5-10})$$

## F6. I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MINOR AXIS

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

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

### 1. Yielding

$$M_n = M_p = F_y Z_y \leq 1.6 F_y S_y \quad (\text{F6-1})$$

## 2. Flange Local Buckling

### 3.

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

**User Note:** All current ASTM A6 W, S, M, C and MC shapes except W21x48, W14x99, W14x90, W12x65, W10x12, W8x31, W8x10, W6x15, W6x9, W6x8.5 and M4x6 have compact flanges at  $F_y = 50$  ksi (345 MPa).

- (b) For sections with noncompact flanges

$$M_n = \left[ M_p - (M_p - 0.7F_y S_y) \left( \frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right] \quad (\text{F6-2})$$

- (c) For sections with slender flanges

$$M_n = F_{cr} S_y \quad (\text{F6-3})$$

where

$$F_{cr} = \frac{0.69E}{\left( \frac{b}{t_f} \right)^2} \quad (\text{F6-4})$$

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

$$\lambda_{pf} = \lambda_p, \text{ the limiting slenderness for a compact flange, Table B4.1}$$

$$\lambda_{rf} = \sqrt{\frac{E}{F_y}}$$

$$b = \text{for flanges of I-shaped members, half the full-flange width, } b_f; \text{ for flanges of channels, the full nominal dimension of the flange, in. (mm)}$$

$$t_f = \text{thickness of the flange, in. (mm)}$$

$$S_y \text{ for a channel shall be taken as the minimum section modulus}$$

## F7. 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 Section B4 for flexure.

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

**User Note:** Very long rectangular HSS bent about the major axis are subject to lateral-torsional buckling; however, the Specification

provides no strength equation for this limit state since beam deflection will control for all reasonable cases.

### 1. Yielding

$$M_n = M_p = F_y Z \quad (\text{F7-1})$$

where

$Z$  = plastic section modulus about the axis of bending, in.<sup>3</sup> (mm<sup>3</sup>)

### 2. Flange Local Buckling

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

(b) For sections with noncompact flanges

$$M_n = M_p - (M_p - F_y S) \left( 3.57 \frac{b}{t_f} \sqrt{\frac{F_y}{E}} - 4.0 \right) \leq M_p \quad (\text{F7-2})$$

(c) For sections with slender flanges

$$M_n = F_y S_e \quad (\text{F7-3})$$

where

$S_e$  = *effective section modulus* determined with the *effective width*,  $b_e$ , of the compression flange taken as:

$$b_e = 1.92 t_f \sqrt{\frac{E}{F_y}} \left[ 1 - \frac{0.38}{b/t_f} \sqrt{\frac{E}{F_y}} \right] \leq b \quad (\text{F7-4})$$

### 3. Web Local Buckling

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

(b) For sections with noncompact webs

$$M_n = M_p - (M_p - F_y S_x) \left( 0.305 \frac{h}{t_w} \sqrt{\frac{F_y}{E}} - 0.738 \right) \leq M_p \quad (\text{F7-5})$$

## F8. ROUND HSS

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

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

### 1. Yielding



$$M_n = M_p = F_y Z \quad (\text{F8-1})$$

## 2. Local Buckling

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

(b) For noncompact sections

$$M_n = \left( \frac{0.021E}{\frac{D}{t}} + F_y \right) S \quad (\text{F8-2})$$

(c) For sections with slender walls

$$M_n = F_{cr} S \quad (\text{F8-3})$$

where

$$F_{cr} = \frac{0.33E}{\frac{D}{t}} \quad (\text{F8-4})$$

$S$  = elastic section modulus, in.<sup>3</sup> (mm<sup>3</sup>)

$t$  = thickness of wall

## F9. TEES AND DOUBLE ANGLES LOADED IN THE PLANE OF SYMMETRY

This section applies to tees and double angles loaded in the plane of symmetry.

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

### 1. Yielding

$$M_n = M_p \quad (\text{F9-1})$$

where

$$M_p = F_y Z_x \leq 1.6 M_y \text{ for stems in tension} \quad (\text{F9-2})$$

$$M_p = F_y Z_x \leq M_y \text{ for stems in compression} \quad (\text{F9-3})$$

### 2. Lateral-Torsional Buckling

$$M_n = M_{cr} = \frac{\pi \sqrt{EI_y GJ}}{L_b} \left[ B + \sqrt{1 + B^2} \right] \quad (\text{F9-4})$$

where

$$B = \pm 2.3 \left( \frac{d}{L_b} \right) \sqrt{\frac{I_y}{J}} \quad (\text{F9-5})$$

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.

### 3. Flange Local Buckling of Tees

(a) For sections with a compact flange in flexural compression, the *limit state of flange local buckling* does not apply.

(b) For sections with a noncompact flange in flexural compression,

$$M_n = M_p - (M_p - 0.7F_y S_{xc}) \left( \frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \leq 1.6M_y \quad (\text{F9-6})$$

(c) For sections with a slender flange in flexural compression

$$M_n = \frac{0.7ES_{xc}}{\left( \frac{b_f}{2t_f} \right)^2} \quad (\text{F9-7})$$

where

$S_{xc}$  = the elastic section modulus referred to the compression flange, in.<sup>3</sup> (mm<sup>3</sup>)

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

$\lambda_{pf} = \lambda_p$ , the limiting slenderness for a compact flange, Table B4.1

$\lambda_{rf} = \lambda_r$ , the limiting slenderness for a noncompact flange, Table B4.1

### 4. Local Buckling of Tee Stems in Flexural Compression

$$M_n = F_{cr} S_x \quad (\text{F9-8})$$

$S_x$  = the elastic section modulus

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

$$\text{When } \frac{d}{t_w} \leq 0.84 \sqrt{\frac{E}{F_y}}$$

$$F_{cr} = F_y \quad (\text{F9-9})$$

$$\text{When } 0.84 \sqrt{\frac{E}{F_y}} < \frac{d}{t_w} \leq 1.03 \sqrt{\frac{E}{F_y}}$$

$$F_{cr} = \left[ 2.55 - 1.84 \frac{d}{t_w} \sqrt{\frac{F_y}{E}} \right] F_y \quad (\text{F9-10})$$

$$\text{When } \frac{d}{t_w} > 1.03 \sqrt{\frac{E}{F_y}}$$

$$F_{cr} = \frac{0.69E}{\left( \frac{d}{t_w} \right)^2} \quad (\text{F9-11})$$

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## F10. SINGLE ANGLES

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

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

If the moment resultant has components about both principal axes or the moment is about one principal axis and there is axial load, the combined stress shall be determined using the provisions of Section H2. The provisions in Sections F10.2(i) and (ii) for bending moment about the geometric axis of equal-leg angles shall not be used if the angle has axial load.

**User Note:** For geometric axis design, use section properties computed about the  $x$ - and  $y$ - axis of the angle, parallel and perpendicular to the legs. For principal axis design use section properties computed about the major and minor principal axes of the angle.

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

**User Note:** For bending about the minor axis, only the limit states of

yielding and leg local buckling apply.

## 1. Yielding

$$M_n = 1.5M_y \quad (\text{F10-1})$$

where

$M_y$  = yield moment about the axis of bending, kip-in. (N-mm)

## 2. Lateral-Torsional Buckling

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

(a) When  $M_e \leq M_y$

$$M_n = \left( 0.92 - \frac{0.17M_e}{M_y} \right) M_e \quad (\text{F10-2})$$

(b) When  $M_e > M_y$

$$M_n = \left( 1.92 - 1.17 \sqrt{\frac{M_y}{M_e}} \right) M_y \leq 1.5M_y \quad (\text{F10-3})$$

where

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

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

(a) With maximum compression at the toe

$$M_e = \frac{0.66Eb^4tC_b}{L^2} \left( \sqrt{1 + 0.78 \left( \frac{Lt}{b^2} \right)^2} - 1 \right) \quad (\text{F10-4a})$$

(b) With maximum tension at the toe

$$M_e = \frac{0.66Eb^4tC_b}{L^2} \left( \sqrt{1 + 0.78 \left( \frac{Lt}{b^2} \right)^2} + 1 \right) \quad (\text{F10-4b})$$

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

**User Note:**  $M_n$  may be taken as  $M_y$  for single angles with their vertical leg toe in compression, and having a span-to-depth ratio less than or

equal to  $\frac{1.64E}{F_y} \sqrt{\left( \frac{t}{b} \right)^2 - 1.4 \frac{F_y}{E}}$ .

(ii) For bending moment 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 Equation

F10-4a or F10-4b.

$M_y$  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:

$$M_e = \frac{0.46Eb^2t^2C_b}{L} \quad (\text{F10-5})$$

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

$$M_e = \frac{4.9EI_zC_b}{L^2} \left( \sqrt{\beta_w^2 + 0.052 \left( \frac{Lt}{r_z} \right)^2} + \beta_w \right) \quad (\text{F10-6})$$

where

$C_b$  is computed using Equation F1-1 with a maximum value of 1.5.

$L$  = laterally *unbraced length* of a member, in. (mm)

$I_z$  = minor principal axis moment of inertia, in.<sup>4</sup> (mm<sup>4</sup>)

$r_z$  = radius of gyration for the minor principal axis, in. (mm)

$t$  = angle leg thickness, in. (mm)

$\beta_w$  = 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  $\beta_w$  shall be used.

**User Note:** The equation for  $\beta_w$  and values for common angle sizes are listed in the Commentary.

### 3. Leg Local Buckling

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

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

- (b) For sections with noncompact legs:

$$M_n = F_y S_c \left( 2.43 - 1.72 \left( \frac{b}{t} \right) \sqrt{\frac{F_y}{E}} \right) \quad (\text{F10-7})$$

- (c) For sections with slender legs:

$$M_n = F_{cr} S_c \quad (\text{F10-8})$$

where

$$F_{cr} = \frac{0.71E}{\left(\frac{b}{t}\right)^2} \quad (\text{F10-9})$$

$b$  = outside width of leg in compression, in. (mm)

$S_c$  = elastic section modulus to the toe in compression relative to the axis of bending, in.<sup>3</sup> (mm<sup>3</sup>). 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.

## F11. RECTANGULAR BARS AND ROUNDS

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

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

### 1. Yielding

For rectangular bars 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:

$$M_n = M_p = F_y Z \leq 1.6 M_y \quad (\text{F11-1})$$

### 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:

$$M_n = C_b \left[ 1.52 - 0.274 \left( \frac{L_b d}{t^2} \right) \frac{F_y}{E} \right] M_y \leq M_p \quad (\text{F11-2})$$

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

$$M_n = F_{cr} S_x \leq M_p \quad (\text{F11-3})$$

where

$$F_{cr} = \frac{1.9EC_b}{\frac{L_b d}{t^2}} \quad (\text{F11-4})$$

$t$  = width of rectangular bar parallel to axis of bending,  
 in. (mm)  
 $d$  = depth of rectangular bar, in. (mm)  
 $L_b$  = length between points that are either braced against  
 lateral displacement of the compression region, or be-  
 tween points braced to prevent twist of the cross sec-  
 tion, in. (mm)

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

## F12. UNSYMMETRICAL SHAPES

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

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

$$M_n = F_n S \quad (\text{F12-1})$$

where

$S$  = lowest elastic section modulus relative to the axis of bending,  
 in.<sup>3</sup> (mm<sup>3</sup>)

### 1. Yielding

$$F_n = F_y \quad (\text{F12-2})$$

### 2. Lateral-Torsional Buckling

$$F_n = F_{cr} \leq F_y \quad (\text{F12-3})$$

where

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

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

### 3. Local Buckling

$$F_n = F_{cr} \leq F_y \quad (\text{F12-4})$$

where

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

### F13. PROPORTIONS OF BEAMS AND GIRDERS

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

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

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

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

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

$$M_n = \frac{F_u A_{fn}}{A_{fg}} S_x \quad (\text{F13-1})$$

where

$A_{fg}$  = gross area of tension flange, calculated in accordance with the provisions of Section B4.3a, in.<sup>2</sup> (mm<sup>2</sup>)

$A_{fn}$  = net area of tension flange, calculated in accordance with the provisions of Section B4.3b, in.<sup>2</sup> (mm<sup>2</sup>)

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

#### 2. Proportioning Limits for I-Shaped Members

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

$$0.1 \leq \frac{I_{yc}}{I_y} \leq 0.9 \quad (\text{F13-2})$$

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

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

$$\left( \frac{h}{t_w} \right)_{\max} = 12.0 \sqrt{\frac{E}{F_y}} \quad (\text{F13-3})$$



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

$$\left( \frac{h}{t_w} \right)_{\max} = \frac{0.40E}{F_y} \quad (\text{F13-4})$$

where

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

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

### 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% 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 specified for compression or tension members in Section E6 or D4, respectively. Bolts or welds connecting flange to web shall also be proportioned to transmit to the web any *loads* applied directly to the flange, unless provision is made to transmit such loads by direct bearing.

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 J2.2, J3.8 or B3.10 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 available 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

$$a' = w \quad (F13-5)$$

where

$w$  = width of cover plate, in. (mm)

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

$$a' = 1.5w \quad (F13-6)$$

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

$$a' = 2w \quad (F13-7)$$

#### 4. Built-Up Beams

Where two or more *beams* or channels are used side-by-side to form a flexural member, they shall be connected together in compliance with Section E6.2. When concentrated *loads* are carried from one beam to another or distributed between the beams, *diaphragms* having sufficient *stiffness* to distribute the load shall be welded or bolted between the beams.

## CHAPTER G DESIGN OF MEMBERS FOR SHEAR

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

The chapter is organized as follows:

- G1. General Provisions
- G2. Members with Unstiffened or Stiffened Webs
- G3. Tension Field Action
- G4. Single Angles
- G5. Rectangular HSS and Box Members
- G6. Round HSS
- G7. Weak Axis Shear in Singly and Doubly Symmetric Shapes
- G8. Beams and Girders with Web Openings

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

- H3.3 Unsymmetric sections.
- J4.2 Shear strength of connecting elements.
- J10.6 Web panel zone shear.

### G1. GENERAL PROVISIONS

Two methods of calculating shear strength are presented below. The method presented in Section G2 does not utilize the post *buckling strength* of the member (*tension field action*). The method presented in Section G3 utilizes tension field action.

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

For all provisions in this chapter except Section G2.1(a):

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

### G2. MEMBERS WITH UNSTIFFENED OR STIFFENED WEBS

#### 1. 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

$$V_n = 0.6F_y A_w C_v \quad (\text{G2-1})$$

(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

$$C_v = 1.0 \quad (\text{G2-2})$$

**User Note:** All current ASTM A6 W, S, and HP shapes except W44x230, W40x149, W36x135, W33x118, W30x90, W24x55, W16x26 and W12x14 meet the criteria stated in Section G2.1 (a) for  $F_y = 50$  ksi (345 MPa).

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

$$C_v = 1.0 \quad (\text{G2-3})$$

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

$$C_v = \frac{1.10\sqrt{k_v E / F_y}}{h/t_w} \quad (\text{G2-4})$$

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

$$C_v = \frac{1.51Ek_v}{(h/t_w)^2 F_y} \quad (\text{G2-5})$$

where

- $A_w$  = the overall depth times the web thickness,  $dt_w$ , in.<sup>2</sup> (mm<sup>2</sup>)
- $h$  = for rolled shapes, the clear distance between flanges less the fillet or corner radii, in. (mm)
- = for built-up welded sections, the clear distance between flanges, in. (mm)
- = for built-up bolted sections, the distance between *fastener* lines, in. (mm)
- = for tees, the overall depth, in. (mm)
- $t_w$  = thickness of web, in. (mm)

The web plate shear 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:

$$k_v = 5 + \frac{5}{(a/h)^2} \quad (\text{G2-6})$$

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

where

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

**User Note:** For all ASTM A6 W, S, M and HP shapes except M12.5x12.4, M12.5x11.6, M12x11.8, M12x10.8, M12x10, M10x8 and M10x7.5, when  $F_y = 50$  ksi (345 MPa),  $C_v = 1.0$ .

## 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 Section G2.1 for  $k_v = 5$ .

The moment of inertia,  $I_{st}$ , of transverse stiffeners used to develop the available web shear strength, as provided in Section G2.1, about an axis in the web center for stiffener pairs or about the face in contact with the web plate for single stiffeners, shall meet the following requirement

$$I_{st} \geq b t_w^3 j \quad (\text{G2-7})$$

where

$$j = \frac{2.5}{(a/h)^2} - 2 \geq 0.5 \quad (\text{G2-8})$$

and

$b$  is the smaller of the dimensions  $a$  and  $h$ .

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.

Bolts connecting stiffeners to the girder web shall be spaced not more than 12 in. (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 10 in. (250 mm).

### G3. TENSION FIELD ACTION

#### 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:

- (a) for *end panels* in all members with transverse stiffeners;
- (b) when  $a/h$  exceeds 3.0 or  $[260/(h/t_w)]^2$  ;
- (c) when  $2A_w / (A_{fc} + A_{ft}) > 2.5$  ; or
- (d) when  $h/b_{fc}$  or  $h/b_{ft} > 6.0$

where

- $A_{fc}$  = area of compression flange, in.<sup>2</sup> (mm<sup>2</sup>)
- $A_{ft}$  = area of tension flange, in.<sup>2</sup> (mm<sup>2</sup>)
- $b_{fc}$  = width of compression flange, in. (mm)
- $b_{ft}$  = width of tension flange, in. (mm)

In these cases, the nominal shear strength,  $V_n$ , shall be determined according to the provisions of Section G2.

#### 2. Shear Strength with Tension Field Action

When *tension field action* is permitted according to Section G3.1, the nominal shear strength,  $V_n$ , with tension field action, according to the *limit state* of tension field *yielding*, shall be

- (a) When  $h/t_w \leq 1.10\sqrt{k_v E / F_y}$

$$V_n = 0.6F_y A_w \quad (G3-1)$$

- (b) When  $h/t_w > 1.10\sqrt{k_v E / F_y}$

$$V_n = 0.6F_y A_w \left( C_v + \frac{1 - C_v}{1.15 \sqrt{1 + (a/h)^2}} \right) \quad (\text{G3-2})$$

where

$k_v$  and  $C_v$  are as defined in Section G2.1.

### 3. Transverse Stiffeners

Transverse *stiffeners* subject to *tension field action* shall meet the requirements of Section G2.2 and the following limitations:

$$(1) \quad (b/t)_{st} \leq 0.56 \sqrt{\frac{E}{F_{yst}}} \quad (\text{G3-3})$$

$$(2) \quad I_{st} \geq I_{st1} + (I_{st2} - I_{st1}) \left[ \frac{V_r - V_{c1}}{V_{c2} - V_{c1}} \right] \quad (\text{G3-4})$$

where

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

$F_{yst}$  = *specified minimum yield stress* of the stiffener material, ksi (MPa)

$I_{st}$  = moment of inertia of the transverse stiffeners about an axis in the web center for stiffener pairs, or about the face in contact with the web plate for single stiffeners, in.<sup>4</sup> (mm<sup>4</sup>)

$I_{st1}$  = minimum moment of inertia of the transverse stiffeners required for development of the web shear buckling resistance in Section G2.2, in.<sup>4</sup> (mm<sup>4</sup>)

$I_{st2}$  = minimum moment of inertia of the transverse stiffeners required for development of the full web shear buckling plus the web tension field resistance,  $V_r = V_{c2}$ , in.<sup>4</sup> (mm<sup>4</sup>)

$$= \frac{h^4 \rho_{st}^{1.3}}{40} \left( \frac{F_{yw}}{E} \right)^{1.5} \quad (\text{G3-5})$$

$V_r$  = larger of the *required shear strengths* in the adjacent web panels using LRFD or ASD load combinations, kips (N)

$V_{c1}$  = smaller of the *available shear strengths in the adjacent web panels* with  $V_n$  as defined in Section G2.1, kips (N)

$V_{c2}$  = smaller of the *available shear strengths* in the adjacent web panels with  $V_n$  as defined in Section G3.2, kips (N)

$\rho_{st}$  = the larger of  $F_{yw}/F_{yst}$  and 1.0

2506  $F_{yw}$  = specified minimum yield stress of the web material, ksi  
 2507 (MPa)

#### 2509 **G4. SINGLE ANGLES**

2510 The nominal shear strength,  $V_n$ , of a single angle leg shall be deter-  
 2511 mined using Equation G2-1 and Section G2.1(b) with  $A_w = bt$  where

2512  $b$  = width of the leg resisting the shear force, in. (mm)

2513  $t$  = thickness of angle leg, in. (mm)

2514  $h/t_w = b/t$

2515  $k_v = 1.2$

#### 2518 **G5. RECTANGULAR HSS AND BOX-SHAPED MEMBERS**

2519 The nominal shear strength,  $V_n$ , of rectangular HSS and box members  
 2520 shall be determined using the provisions of Section G2.1 with  $A_w = 2ht$   
 2521 where

2522  $h$  = width resisting the shear force, taken as the clear distance  
 2523 between the flanges less the inside corner radius on each  
 2524 side,

2525  $t$  = design wall thickness, equal to 0.93 times the nominal wall  
 2526 thickness for ERW HSS and equal to the nominal thickness  
 2527 for SAW HSS, in. (mm)

2528  $t_w = t$

2529  $k = 5$

2530 If the corner radius is not known,  $h$  shall be taken as the corresponding  
 2531 outside dimension minus 3 times the thickness.

#### 2534 **G6. ROUND HSS**

2535 The nominal shear strength,  $V_n$ , of round HSS, according to the limit  
 2536 states of shear yielding and shear buckling, shall be determined as:

$$2537 \quad V_n = F_{cr} A_g / 2 \quad (\text{G6-1})$$

2540 where

2541  $F_{cr}$  shall be the larger of

$$2542 \quad F_{cr} = \frac{1.60E}{\sqrt{\frac{L_v}{D} \left( \frac{D}{t} \right)^4}} \quad (\text{G6-2a})$$

2546 and

$$2547 \quad F_{cr} = \frac{0.78E}{\left( \frac{D}{t} \right)^{\frac{3}{2}}} \quad (\text{G6-2b})$$



but shall not exceed  $0.6F_y$

$A_g$  = gross area of section based on design wall thickness, in.<sup>2</sup>  
(mm<sup>2</sup>)

$D$  = outside diameter, in. (mm)

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

**User Note:** The shear buckling equations, Equations G6-2a and G6-2b, will control for  $D/t$  over 100, high strength steels, and long lengths. For standard sections, shear yielding will usually control.

## G7. 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 Equation G2-1 and Section G2.1(b) with  $A_w = b_f t_f$ ,  $h/t_w = b_f/t_f$ , and  $k_v = 1.2$ .

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

## G8. BEAMS AND GIRDERS WITH WEB OPENINGS

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

## CHAPTER H

### DESIGN OF MEMBERS FOR COMBINED FORCES AND TORSION

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

The chapter is organized as follows:

- H1. Doubly and Singly Symmetric Members Subject to Flexure and Axial Force
- H2. Unsymmetric and Other Members Subject to Flexure and Axial Force
- H3. Members Subject to Torsion and Combined Torsion, Flexure, Shear and/or Axial Force
- H4. Rupture of Flanges with Holes Subject to Tension

**User Note:** For *composite* members, see Chapter I.

#### H1. DOUBLY AND SINGLY SYMMETRIC MEMBERS SUBJECT TO FLEXURE AND AXIAL FORCE

##### 1. Doubly and Singly Symmetric Members Subject to Flexure and Compression

The interaction of flexure and compression in doubly symmetric members and singly symmetric members for which  $0.1 \leq (I_{yc}/I_y) \leq 0.9$ , constrained to bend about a *geometric axis* ( $x$  and/or  $y$ ) shall be limited by Equations H1-1a and H1-1b, where  $I_{yc}$  is the moment of inertia about the  $y$ -axis of the compression flange, in.<sup>4</sup> (mm<sup>4</sup>).

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

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

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

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

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

where

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

$P_c$  = available axial strength, kips (N)

$M_r$  = required flexural strength, kip-in. (N-mm)

$M_c$  = available flexural strength, kip-in. (N-mm)

$x$  = subscript relating symbol to *strong axis* bending

$y$  = subscript relating symbol to *weak axis* bending

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

$P_r$  = required axial strength using LRFD load combinations, kips (N)

$P_c = \phi_c P_n$  = design axial strength, determined in accordance with Chapter E, kips (N)

$M_r$  = required flexural strength using LRFD load combinations, kip-in. (N-mm)

$M_c = \phi_b M_n$  = design flexural strength determined in accordance with Chapter F, kip-in. (N-mm)

$\phi_c$  = resistance factor for compression = 0.90

$\phi_b$  = resistance factor for flexure = 0.90

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

$P_r$  = required axial strength using ASD load combinations, kips (N)

$P_c = P_n / \Omega_c$  = allowable axial strength, determined in accordance with Chapter E, kips (N)

$M_r$  = required flexural strength using ASD load combinations, kip-in. (N-mm)

$M_c = M_n / \Omega_b$  = allowable flexural strength determined in accordance with Chapter F, kip-in. (N-mm)

$\Omega_c$  = safety factor for compression = 1.67

$\Omega_b$  = safety factor for flexure = 1.67

## 2. Doubly and Singly Symmetric Members Subject to Flexure and Tension

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

where

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

$P_r$  = required tensile strength using LRFD load combinations, kips (N)  
 $P_c = \phi_t P_n$  = design tensile strength, determined in accordance with Section D2, kips (N)  
 $M_r$  = required flexural strength using LRFD load combinations, kip-in. (N-mm)  
 $M_c = \phi_b M_n$  = design flexural strength determined in accordance with Chapter F, kip-in. (N-mm)  
 $\phi_t$  = resistance factor for tension (see Section D2)  
 $\phi_b$  = resistance factor for flexure = 0.90

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

$P_r$  = required tensile strength using ASD load combinations, kips (N)  
 $P_c = P_n / \Omega_t$  = allowable tensile strength, determined in accordance with Section D2, kips (N)  
 $M_r$  = required flexural strength using ASD load combinations, kip-in. (N-mm)  
 $M_c = M_n / \Omega_b$  = allowable flexural strength determined in accordance with Chapter F, kip-in. (N-mm)  
 $\Omega_t$  = safety factor for tension (see Section D2)  
 $\Omega_b$  = safety factor for flexure = 1.67

For doubly symmetric members,  $C_b$  in Chapter F may be multiplied

by  $\sqrt{1 + \frac{\alpha P_r}{P_{ey}}}$  for axial tension that acts concurrently with flexure,

where

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

and

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

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

**3. Doubly Symmetric Rolled Compact Members Subject to Single Axis Flexure and Compression**

For doubly symmetric rolled compact members with  $(KL)_z \leq (KL)_y$  subjected to flexure and compression with moments primarily about their major axis, it is permissible to consider the two independent *limit states*, *in-plane instability* and *out-of-plane buckling* or *lateral-torsional buckling*, separately in lieu of the combined approach

provided in Section H1.1.

- (a) For the limit state of in-plane instability, Equations H1-1 shall be used with  $P_{cy}$ ,  $M_{rx}$  and  $M_{cx}$  determined in the plane of bending.
- (b) For the limit state of out-of-plane buckling and lateral-torsional buckling:

$$1.5 \frac{P_r}{P_{cy}} - 0.5 \left( \frac{P_r}{P_{cy}} \right)^2 + \left( \frac{M_{rx}}{C_b M_{cx}} \right)^2 \leq 1.0 \quad (\text{H1-2})$$

where

$P_{cy}$  = available compressive strength out of the plane of bending, kips (N)

$C_b$  = lateral-torsional buckling modification factor determined from Section F1

$M_{cx}$  = available lateral-torsional strength for strong axis flexure determined from Chapter F, kip-in. (N-mm)

**User Note:** In Equation H1-2,  $C_b M_{cx}$  may be larger than  $\phi_b M_{px}$  in LRFD or  $M_{px}/\Omega_b$  in ASD. The yielding resistance of the beam-column is captured by Equations H1-1.

For members with  $M_{ry}/M_{cy} \geq 0.05$ , the provisions of Section H1.1 shall be followed.

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

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

$$\left| \frac{f_{ra}}{F_{ca}} + \frac{f_{rbw}}{F_{cbw}} + \frac{f_{rbz}}{F_{cbz}} \right| \leq 1.0 \quad (\text{H2-1})$$

where

$f_{ra}$  = required axial stress at the point of consideration using LRFD or ASD load combinations, ksi (MPa)

$F_{ca}$  = available axial stress at the point of consideration, ksi (MPa)

$f_{rbw}, f_{rbz}$  = required flexural stress at the point of consideration, ksi (MPa)

$F_{cbw}, F_{cbz}$  = available flexural stress at the point of consideration about the major/minor axis, ksi (MPa)

$w$  = subscript relating symbol to major principal axis bending

$z$  = subscript relating symbol to minor principal axis bending

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

- $f_{ra}$  = required axial stress at the point of consideration using LRFD load combinations, ksi (MPa)
- $F_{ca}$  =  $\phi_c F_{cr}$  = *design axial stress*, determined in accordance with Chapter E for compression or Section D2 for tension, ksi (MPa)
- $f_{rbw}, f_{rbz}$  = required flexural stress at the point of consideration (major axis, minor axis) using LRFD or ASD load combinations, ksi (MPa)
- $F_{cbw}, F_{cbz}$  =  $\frac{\phi_b M_n}{S}$  = *design flexural stress* determined in accordance with Chapter F, ksi (MPa). Use the section modulus for the specific location in the cross section and consider the sign of the stress.
- $\phi_c$  = *resistance factor* for compression = 0.90
- $\phi_t$  = *resistance factor* for tension (Section D2)
- $\phi_b$  = *resistance factor* for flexure = 0.90

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

- $f_{ra}$  = required axial stress at the point of consideration using ASD load combinations, ksi (MPa)
- $F_{ca}$  =  $\frac{F_{cr}}{\Omega_c}$  = *allowable axial stress* determined in accordance with Chapter E for compression, or Section D2 for tension, ksi (MPa)
- $f_{rbw}, f_{rbz}$  = required flexural stress at the point of consideration (major axis, minor axis) using LRFD or ASD load combinations, ksi (MPa)
- $F_{cbw}, F_{cbz}$  =  $\frac{M_n}{\Omega_b S}$  = *allowable flexural stress* determined in accordance with Chapter F, ksi (MPa). Use the section modulus for the specific location in the cross section and consider the sign of the stress.
- $\Omega_c$  = *safety factor* for compression = 1.67
- $\Omega_t$  = *safety factor* for tension (see Section D2)
- $\Omega_b$  = *safety factor* for flexure = 1.67

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

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

### H3. MEMBERS SUBJECT TO TORSION AND COMBINED TORSION, FLEXURE, SHEAR AND/OR AXIAL FORCE

#### 1. Round and Rectangular HSS Subject to Torsion

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

$$\phi_T = 0.90 \text{ (LRFD)} \quad \Omega_T = 1.67 \text{ (ASD)} \quad T_n = F_{cr} C \quad (\text{H3-1})$$

where

$C$  is the HSS torsional constant

The critical stress,  $F_{cr}$ , shall be determined as follows:

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

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

and

$$(ii) \quad F_{cr} = \frac{0.60E}{\left( \frac{D}{t} \right)^2} \quad (\text{H3-2b})$$

but shall not exceed  $0.6F_y$ ,

where

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

$D$  = outside diameter, in. (mm)

(b) For rectangular HSS

(i) When  $h/t \leq 2.45\sqrt{E/F_y}$

$$F_{cr} = 0.6F_y \quad (\text{H3-3})$$

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

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

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

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

$h$  = as defined in Section B4.1b(d) for the longer side, the clear distance between the sides less the inside corner radius on each side, in. (mm)  
 $t$  = design wall thickness defined in Section B4.2, in. (mm)

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

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

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

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

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

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

where

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

$P_r$  = *required axial strength* using LRFD load combinations, kips (N)

$P_c$  =  $\phi P_n$ , *design tensile or compressive strength* in accordance with Chapter D or E, kips (N)

$M_r$  = *required flexural strength* using LRFD load combinations, kip-in. (N-mm)

$M_c$  =  $\phi_b M_n$ , *design flexural strength* in accordance with Chapter F, kip-in. (N-mm)

$V_r$  = *required shear strength* using LRFD load combinations, kips (N)

$V_c$  =  $\phi_v V_n$ , *design shear strength* in accordance with Chapter G, kips (N)



$T_r$  = required torsional strength using LRFD load combinations,  
kip-in. (N-mm)

$T_c = \phi_T T_n$ , *design torsional strength* in accordance with Section  
H3.1, kip-in. (N-mm)

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

$P_r$  = required axial strength using *ASD load combinations*, kips  
(N)

$P_c = P_n / \Omega$ , *allowable tensile or compressive strength* in accor-  
dance with Chapter D or E, kips (N)

$M_r$  = required flexural strength using ASD load combinations  
determined in accordance with Section B5, kip-in. (N-mm)

$M_c = M_n / \Omega_b$ , *allowable flexural strength* in accordance with  
Chapter F, kip-in. (N-mm)

$V_r$  = required shear strength using ASD load combinations, kips  
(N)

$V_c = V_n / \Omega_v$ , *allowable shear strength* in accordance with Chap-  
ter G, kips (N)

$T_r$  = required torsional strength using ASD load combinations,  
kip-in. (N-mm)

$T_c = T_n / \Omega_T$ , *allowable torsional strength* in accordance with  
Section H3.1, kip-in. (N-mm)

**3. Non-HSS Members Subject to Torsion and Combined Stress**

The *available torsional strength* for non-HSS members shall be the  
lowest value obtained according to the *limit states* of *yielding* under  
normal stress, *shear yielding* under shear stress, or *buckling*, deter-  
mined 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

$$F_n = F_y \text{ (H3-7)}$$

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

$$F_n = 0.6F_y \text{ (H3-8)}$$

(c) For the limit state of buckling

$$F_n = F_{cr} \text{ (H3-9)}$$

where

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

ksi (MPa)

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

#### H4. RUPTURE OF FLANGES WITH HOLES SUBJECT TO TENSION

At locations of bolt holes in flanges subject to tension under combined axial force and major axis flexure, flange *tensile rupture strength* shall be limited by Equation H4-1. Each flange subject to tension due to axial force and flexure shall be checked separately.

$$\frac{P_r}{P_c} + \frac{M_{rx}}{M_{cx}} \leq 1.0 \quad (\text{H4-1})$$

where

$P_r$  = required axial strength of the member at the location of the bolt holes, positive in tension, negative in compression, kips (N)

$P_c$  = available axial strength for the limit state of tensile rupture of the net section at the location of bolt holes, kips (N)

$M_{rx}$  = required flexural strength at the location of the bolt holes; positive for tension in the flange under consideration, negative for compression, kip-in. (N-mm)

$M_{cx}$  = available flexural strength about x-axis for the limit state of tensile rupture of the flange, determined according to Section F13.1. When the limit state of tensile rupture in flexure does not apply, use the plastic bending moment,  $M_p$ , determined with bolt holes not taken into consideration, kip-in. (N-mm)

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

$P_r$  = required axial strength using LRFD *load combinations*, kips (N)

$P_c = \phi_t P_n$  = design axial strength for the limit state of tensile rupture, determined in accordance with Section D2(b), kips (N)

$M_{rx}$  = required flexural strength using LRFD *load combinations*, kip-in. (N-mm)

$M_{cx} = \phi_b M_n$  = design flexural strength determined in accordance with Section F13.1 or the plastic bending moment,  $M_p$ , determined with bolt holes not taken into consideration, as applicable, kip-in. (N-mm)

$\phi_t$  = *resistance factor* for tensile rupture = 0.75

$\phi_b$  = *resistance factor* for flexure = 0.90

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

$P_r$  = required axial strength using *ASD load combinations*,  
kips (N)

$P_c = P_n / \Omega_t$  = allowable axial strength for the limit state of  
tensile rupture, determined in accordance with Section  
D2(b), kips (N)

$M_{rx}$  = required flexural strength using ASD load combina-  
tions, kip-in. (N-mm)

$M_{cx} = M_n / \Omega_b$  = allowable flexural strength determined in  
accordance with Section F13.1, or the plastic bending  
moment,  $M_p$ , determined with bolt holes not taken into  
consideration, as applicable, kip-in. (N-mm)

$\Omega_t$  = *safety factor* for tensile rupture = 2.00

$\Omega_b$  = *safety factor* for flexure = 1.67

DRAFT

## CHAPTER I DESIGN OF COMPOSITE MEMBERS

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

The chapter is organized as follows:

- I1. General Provisions
- I2. Axially Loaded Members
- I3. Flexural Members
- I4. Combined Axial Force and Flexure
- I5. Load Transfer
- I6. Composite Diaphragms and Collector Beams
- I7. Steel Anchors
- I8. Special Cases

### **I1. GENERAL PROVISIONS**

In determining *load effects* in members and *connections* of a structure that includes *composite* members, consideration shall be given to the effective sections at the time each increment of *load* is applied.

#### **1. Concrete and Steel Reinforcement**

The design, detailing and material properties related to the concrete and reinforcing steel portions of composite construction shall comply with the reinforced concrete and reinforcing bar design specifications stipulated by the *applicable building code*. Additionally, the provisions in ACI 318 shall apply with the following exceptions and limitations:

- (1) ACI 318 Sections 7.8.2 and 10.13, and Chapter 21 shall be excluded in their entirety.

**User Note:** It is the intent of the Specification that the concrete and reinforcing steel portions of composite concrete members be detailed utilizing the non-composite provisions of ACI 318 as modified by the Specification. All requirements specific to composite members are covered in the Specification.

- (2) Concrete and steel reinforcement material limitations shall be as specified in Section I1.3.
- (3) Concrete and steel reinforcement components designed in accordance with ACI 318 shall be based on a level of loading corresponding to *LRFD load combinations*. This provision shall also apply to steel headed stud anchors designed in accordance with ACI 318 Appendix D where required by Section I7.3

- (4) Transverse reinforcement limitations shall be as specified in Section I2.1a(2), in addition to those specified in ACI 318.
- (5) The minimum longitudinal reinforcing ratio for encased composite members shall be as specified in Section I2.1a(3).

## 2. Nominal Strength of Composite Sections

The *nominal strength* of composite sections shall be determined in accordance with the *plastic stress distribution method* or the *strain compatibility method* as defined in this section.

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

### 2a. Plastic Stress Distribution Method

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

### 2b. Strain Compatibility Method

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

**User Note:** The strain compatibility method should be used to determine nominal strength for irregular sections and for cases where the steel does not exhibit elasto-plastic behavior. General guidelines for the strain compatibility method for encased members subjected to axial load, flexure or both are given in AISC Design Guide 6 and ACI 318.

## 3. Material Limitations

For concrete, structural steel and steel reinforcing bars in composite systems, the following limitations shall be met, unless justified by testing or analysis:

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

**User Note:** Higher strength concrete material properties may be

used for *stiffness* calculations but may not be relied upon for strength calculations unless justified by testing or analysis.

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

#### 4. Limitations on Filled Composite Sections for Local Buckling

For filled composite sections, the following limitations shall be met:

- (1) For filled sections with wall slenderness below the  $\lambda_p$  local buckling limits in Table I1.1, the axial and bending strength shall be based on the plastic capacity of the section or an elastic stress superposition method for flexure as defined in Section I3.3a(a).
- (2) For filled sections having wall slenderness between  $\lambda_p$  and  $\lambda_r$ , the cross-sectional strength shall be taken as a linear interpolation between the plastic capacity ( $\lambda_p$ ) and that at first yield ( $\lambda_r$ ). The capacity at first yield shall be calculated based on assuming an elastic stress distribution for the compression block with the concrete compressive stress at  $0.70f'_c$  and the extreme steel fiber in tension at yield.
- (3) For filled sections with slenderness larger than  $\lambda_r$ , the strength shall be calculated based on an elastic distribution but with the steel compressive stress limited to the critical stress given below:

- (a) For uniform compression in flanges of rectangular filled sections:

$$\text{When } 3 \sqrt{\frac{E}{F_y}} \leq b/t \leq 7 \sqrt{\frac{E}{F_y}}$$

$$F_{cr} = 9 E / (b/t)^2 \quad (\text{I1-1})$$

- (b) For uniform compression in round filled sections:

$$\text{When } 0.19 E/F_y \leq D/t \leq 0.78 E/F_y$$

$$F_{cr} = 0.7 F_y / [(D/t)(F_y/E)]^{0.2} \quad (\text{I1-2})$$

- (c) For flexure in round filled sections:

$$\text{When } D/t \geq 0.39 E/F_y$$

$$F_{cr} = 0.39 E / (D/t) \quad (\text{I1-3})$$

TABLE I1.1

Limiting Width-Thickness Ratios for Composite Compression Elements

Section	Width-Thickness Ratio	$\lambda_p$ (Compact)	$\lambda_r$ (Noncompact)
Uniform compression in flanges of rectangular filled HSS	$b/t$	$2.26 (E/F_y)^{0.5}$	$3.00 (E/F_y)^{0.5}$
Flexure in webs of rectangular filled HSS	$h/t$	$3.00 (E/F_y)^{0.5}$	$7.00 (E/F_y)^{0.5}$
Uniform compression in round filled sections	$D/t$	$0.15 E/F_y$	$0.19 E/F_y$
Flexure in round filled sections	$D/t$	$0.09 E/F_y$	$0.39 E/F_y$

## 12. AXIALLY LOADED MEMBERS

This section shall apply to two types of *composite* axially loaded members: encased composite members and filled composite members.

### 1. Encased Composite Members

#### 1a. Limitations

For encased *composite* members, the following limitations shall be met:

- (1) The cross-sectional area of the steel core shall comprise at least 1% of the total composite cross section.
- (2) Concrete encasement of the steel core shall be reinforced with continuous longitudinal bars and lateral ties or spirals. Where lateral ties are used, a minimum of either a No. 3 (10 mm) bar spaced at a maximum of 12 in. (305 mm) on center, or a No. 4 (13 mm) bar or larger spaced at a maximum of 16 in. (406 mm) on center shall be used. Deformed wire or welded wire reinforcement of equivalent area are permitted. Maximum spacing of lateral ties shall not exceed 0.5 times the least column dimension.
- (3) The minimum reinforcement ratio for continuous longitudinal reinforcing,  $\rho_{sr}$ , shall be 0.004, where  $\rho_{sr}$  is given by:

$$\rho_{sr} = \frac{A_{sr}}{A_g} \quad (I2-1)$$

where

$$\begin{aligned} A_{sr} &= \text{area of continuous reinforcing bars, in.}^2 \text{ (mm}^2\text{)} \\ A_g &= \text{gross area of composite member, in.}^2 \text{ (mm}^2\text{)} \end{aligned}$$

**User Note:** Refer to Sections 7.10 and 10.9.3 of ACI 318 for additional tie and spiral reinforcing provisions.

## 1b. Compressive Strength

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

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

(a) When  $\frac{P_{no}}{P_e} \leq 2.25$

$$P_n = P_{no} \left[ .658^{\frac{P_{no}}{P_e}} \right] \quad (12-2)$$

(b) When  $\frac{P_{no}}{P_e} > 2.25$

$$P_n = 0.877 P_e \quad (12-3)$$

where

$$P_{no} = A_s F_y + A_{sr} F_{ysr} + 0.85 A_c f'_c \quad (12-4)$$

$$P_e = \text{elastic critical buckling load} \\ = \pi^2 (EI_{eff}) / (KL)^2 \quad (12-5)$$

$$A_c = \text{area of concrete, in.}^2 \text{ (mm}^2\text{)}$$

$$A_s = \text{area of the steel section, in.}^2 \text{ (mm}^2\text{)}$$

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

$$EI_{eff} = \text{effective stiffness of composite section, kip-in.}^2 \text{ (N-mm}^2\text{)} \\ = E_s I_s + 0.5 E_s I_{sr} + C_1 E_c I_c \quad (12-6)$$

$$C_1 = \text{coefficient for calculation of effective rigidity of an} \\ \text{encased composite compression member}$$

$$= 0.1 + 2 \left( \frac{A_s}{A_c + A_s} \right) \leq 0.3 \quad (12-7)$$

$$E_s = \text{modulus of elasticity of steel} = 29,000 \text{ ksi (210 MPa)}$$

$$f'_c = \text{specified compressive strength of concrete, ksi (MPa)}$$

$$F_y = \text{specified minimum yield stress of steel section, ksi (MPa)}$$

$$F_{ysr} = \text{specified minimum yield stress of reinforcing bars, ksi} \\ \text{(MPa)}$$

$$I_c = \text{moment of inertia of the concrete section about the plastic} \\ \text{centroid of the composite section, in.}^4 \text{ (mm}^4\text{)}$$



$I_s$  = moment of inertia of steel shape about the plastic centroid of the composite section, in.<sup>4</sup> (mm<sup>4</sup>)

$I_{sr}$  = moment of inertia of reinforcing bars about the plastic centroid of the composite section, in.<sup>4</sup> (mm<sup>4</sup>)

$K$  = effective length factor

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

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

The *available compressive strength* need not be less than that specified for the bare steel member as required by Chapter E.

**User Note:** Equations I2-6 and I2-10 were developed based on a data base of symmetric composite sections. See commentary for guidelines.

### 1c. Tensile Strength

The *available tensile strength* of axially loaded encased composite members shall be determined for the limit state of *yielding* as follows:

$$P_n = A_s F_y + A_{sr} F_{ysr} \quad (I2-8)$$

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

### 1d. Shear Strength

The *available shear strength* shall be calculated based on one of the following:

(a) The shear strength of the steel section alone as specified in Chapter G.

(b) The shear strength of the reinforced concrete portion (concrete plus steel reinforcement) alone as defined by ACI 318 with

$$\phi_v = 0.75 \text{ (LRFD)} \quad \Omega_v = 2.00 \text{ (ASD)}$$

(c) The shear strength of the steel section as defined in Section G plus the reinforcing steel as defined by ACI 318 with

$$\phi_v = 0.75 \text{ (LRFD)} \quad \Omega_v = 2.00 \text{ (ASD)}$$

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

### 1e. Load Transfer

Load transfer requirements for encased composite members shall be determined in accordance with Section I5.

## 1f. Detailing Requirements

Clear spacing between the steel core and longitudinal reinforcing shall be a minimum of 1.5 reinforcing bar diameters, but not less than 1.5 in. (38 mm).

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

## 2. Filled Composite Members

### 2a. Limitations

For filled composite members, the cross-sectional area of the steel section shall comprise at least 1% of the total composite cross section.

### 2b. Compressive Strength

The available compressive strength of axially loaded filled composite members shall be determined for the *limit state of flexural buckling* in accordance with Section I2.1b with the following modifications:

For filled sections with slenderness less than or equal to  $\lambda_p$

$$P_{no} = A_s F_y + A_{sr} F_{ysr} + C_2 A_c f'_c \quad (I2-9a)$$

For filled sections with wall slenderness greater than or equal to  $\lambda_r$

$$P_{no} = A_s F_{cr} + 0.7 f'_c (A_c + A_{sr} E_s / E_c) \quad (I2-9b)$$

where

$\lambda_p$  and  $\lambda_r$  are defined in Table I1.1

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

$$EI_{eff} = E_s I_s + E_s I_{sr} + C_3 E_c I_c \quad (I2-10)$$

$C_3$  = coefficient for calculation of effective rigidity of a filled composite compression member

$$= 0.6 + 2 \left( \frac{A_s}{A_c + A_s} \right) \leq 0.9 \quad (I2-11)$$

$F_{cr}$  is obtained using Equations I1-1 to I1-3

Linear interpolation between  $P_{no}$  from Equations I2-9a and I2-9b shall be used for sections with wall slenderness between  $\lambda_p$  and  $\lambda_r$ .

The available compressive *strength* need not be less than specified for the bare steel member as required by Chapter E.

## 2c. Tensile Strength

The available tensile strength of axially loaded filled composite members shall be determined for the limit state of *yielding* as follows:

$$P_n = A_s F_y + A_{sr} F_{ysr} \quad (I2-12)$$

$$\phi_t = 0.90 \text{ (LRFD)}$$

$$\Omega_t = 1.67 \text{ (ASD)}$$

## 2d. Shear Strength

The available shear strength shall be determined based on one of the following:

(1) The shear strength of the steel section alone as specified in Chapter G.

(2) The shear strength of the reinforced concrete portion (concrete plus steel reinforcement) alone as defined by ACI 318 with

$$\phi_v = 0.75 \text{ (LRFD)}$$

$$\Omega_v = 2.00 \text{ (ASD)}$$

(3) The shear strength of the steel section as defined in Section G plus the reinforcing steel as defined by ACI 318 with

$$\phi_v = 0.75 \text{ (LRFD)}$$

$$\Omega_v = 2.00 \text{ (ASD)}$$

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

## 2e. Load Transfer

Load transfer requirements for filled composite members shall be determined in accordance with Section I5.

# I3. FLEXURAL MEMBERS

This section applies to three types of *composite* flexural members: composite *beams* with *steel anchors* consisting of steel *headed stud anchors* or *steel channel anchors*, concrete-encased members, and filled members.

## 1. General

### 1a. Effective Width

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

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

## 1b. Strength during Construction

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

## 2. Composite Beams with Steel Headed Stud or Steel Channel Anchors

### 2a. Positive Flexural Strength

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

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

- (a) When  $h / t_w \leq 3.76 \sqrt{E / F_y}$

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

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

- (b) When  $h / t_w > 3.76 \sqrt{E / F_y}$

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

### 2b. Negative Flexural Strength

The *available negative flexural strength* shall be determined for the steel section alone, in accordance with the requirements of Chapter F.

Alternatively, the available negative flexural strength shall be

determined from the plastic stress distribution on the composite section, for the *limit state of yielding (plastic moment)*, with

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

provided that the following limitations are met:

- (1) The steel beam is *compact* and is adequately braced in accordance with Chapter F.
- (2) Steel headed stud or steel channel anchors connect the slab to the steel beam in the negative moment region.
- (3) The slab reinforcement parallel to the steel beam, within the effective width of the slab, is *properly developed*.

## 2c. Composite Beams with Formed Steel Deck

### (1) General

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

- (1) The *nominal rib height* shall not be greater than 3 in. (75 mm). The average width of concrete rib or haunch,  $w_r$ , shall be not less than 2 in. (50 mm), but shall not be taken in calculations as more than the minimum clear width near the top of the steel deck.
- (2) The concrete slab shall be connected to the steel beam with welded steel headed stud anchors,  $\frac{3}{4}$  in. (19 mm) or less in diameter (AWS D1.1). Steel headed stud anchors shall be welded either through the deck or directly to the steel cross section. Steel headed stud anchors, after installation, shall extend not less than  $1\frac{1}{2}$  in. (38 mm) above the top of the steel deck and there shall be at least  $\frac{1}{2}$  in. (13 mm) of concrete cover above the top of the installed steel headed stud anchors prior to the placement of concrete.
- (3) The slab thickness above the steel deck shall be not less than 2 in. (50 mm).
- (4) Steel deck shall be anchored to all supporting members at a spacing not to exceed 18 in. (460 mm). Such anchorage shall be provided by steel headed stud anchors, a combination of steel headed stud anchors and arc spot (puddle) welds, or other devices specified by the contract documents.

**(2) Deck Ribs Oriented Perpendicular to Steel Beam**

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

**(3) Deck Ribs Oriented Parallel to Steel Beam**

Concrete below the top of the steel deck is permitted to be included in determining composite section properties and shall be included in calculating  $A_c$ .

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

When the nominal depth of steel deck is 1½ in. (38 mm) or greater, the average width,  $w_r$ , of the supported haunch or rib shall be not less than 2 in. (50 mm) for the first steel headed stud anchor in the transverse row plus four stud diameters for each additional steel headed stud anchor.

**2d. Load Transfer between Steel Beam and Concrete Slab**

**(1) Load Transfer for Positive Flexural Strength**

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

(a) Concrete crushing

$$V'_r = 0.85f'_cA_c \quad (\text{I3-1a})$$

(b) Tensile yielding of the steel section

$$V'_r = F_yA_s \quad (\text{I3-1b})$$

(c) Shear strength of steel headed stud or steel channel anchors

$$V'_r = \sum Q_n \quad (\text{I3-1c})$$

where

$A_c$  = area of concrete slab within effective width, in.<sup>2</sup> (mm<sup>2</sup>)

$A_s$  = area of steel cross section, in.<sup>2</sup> (mm<sup>2</sup>)  
 $\Sigma Q_n$  = sum of *nominal shear strengths* of steel headed stud or steel channel anchors between the point of maximum positive moment and the point of zero moment, kips (N)

## (2) Load Transfer for Negative Flexural Strength

In continuous composite beams where longitudinal reinforcing steel in the negative moment regions is considered to act compositely with the steel beam, the total horizontal shear between the point of maximum negative moment and the point of zero moment shall be determined as the lower value in accordance with the following limit states:

(a) For the limit state of tensile yielding of the slab reinforcement

$$V'_r = A_{sr} F_{ysr} \quad (I3-2a)$$

where

$A_{sr}$  = area of adequately developed longitudinal reinforcing steel within the effective width of the concrete slab, in.<sup>2</sup> (mm<sup>2</sup>)

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

(b) For the limit state of shear strength of steel headed stud or steel channel anchors

$$V'_r = \Sigma Q_n \quad (I3-2b)$$

## 2e. Shear Strength

The available shear strength of composite beams with steel headed stud or steel channel anchors shall be determined based upon the properties of the steel section alone in accordance with Chapter G.

## 3. Concrete-Encased and Filled Members

### 3a. Flexural Strength

The *available flexural strength* of concrete-encased and filled members shall be determined as follows:

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

The nominal flexural strength,  $M_n$ , shall be determined using one of the following methods:

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

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

(c) The plastic stress distribution on the composite section or the strain-compatibility method, for the limit state of yielding (plastic moment) on the composite section. For concrete-encased members, steel anchors shall be provided.

For filled sections, the nominal flexural strength,  $M_n$ , shall be determined using  $F_y$  or  $F_{cr}$  as defined in Section I1.4, whichever is applicable.

### 3b. Shear Strength

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

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

## I4. COMBINED AXIAL FORCE AND FLEXURE

For composite members with wall slenderness ( $b/t$  or  $D/t$ ) less than or equal to  $\lambda_p$ , the interaction between axial forces and flexure in composite members shall be based on the interaction equations of Section H1.1 or one of the methods as defined in Section I1.2. For filled sections with wall slenderness ( $b/t$  or  $D/t$ ) greater than  $\lambda_p$ , the interaction between axial forces and flexure shall be based on the interaction equations of Section H1.1. The interaction between axial forces and flexure in composite members shall account for stability as required by Chapter C. The *available compressive strength* and the *available flexural strength* shall be determined as defined in Sections I2 and I3, respectively. To account for the influence of length effects on the axial strength of the member, the nominal axial strength of the member shall be determined in accordance with Section I2.

**User Note:** Methods for determining the capacity of composite beam-columns can be found in the Commentary.

## I5. LOAD TRANSFER

### 1. General Requirements

When external forces are applied to an axially loaded encased or filled



composite member, the introduction of force to the member and the transfer of longitudinal shears within the member shall be assessed in accordance with the requirements for load allocation presented in this section.

The *design strength*,  $\phi R_n$ , or the *allowable strength*,  $R_n/\Omega$ , of the applicable force transfer mechanisms as determined in accordance with Section 15.3 shall equal or exceed the required longitudinal shear force to be transferred,  $V'_r$ , as determined in accordance with Section 15.2.

## 2. Load Allocation

Load allocation shall be determined based upon the distribution of external load in accordance with the following requirements:

**User Note:** Bearing strength provisions for externally applied forces are located in Section J8. For filled composite members, the term  $\sqrt{A_2/A_1}$  in Equation J8-2 may be taken equal to 2.0 due to confinement effects.

### 2a. External Force Applied to Steel Section

When the entire external *force* is applied directly to the steel section, the force required to be transferred to the concrete,  $V'_r$ , shall be determined as follows:

$$V'_r = P_r(1 - A_s F_y / P_{no}) \quad (I5-1)$$

where

$P_r$  = external design force applied to the composite member, kips (N)

$P_{no}$  = nominal axial compressive strength without consideration of *length effects*, determined by Equation I2-4, kips (N)

### 2b. External Force Applied to Concrete

When the entire external force is applied directly to the concrete encasement or concrete fill, the force required to be transferred to the steel,  $V'_r$ , shall be determined as follows:

$$V'_r = P_r (A_s F_y / P_{no}) \quad (I5-2)$$

### 2c. External Force Applied Concurrently to Steel and Concrete

When the external force is applied concurrently to the steel section and concrete encasement or concrete fill,  $V'_r$  shall be determined as the force required to establish equilibrium of the cross section.

**User Note:** The Commentary provides an acceptable method of

determining the longitudinal shear force required for equilibrium of the cross section.

### 3. Force Transfer Mechanisms

The nominal strength,  $R_n$ , of the force transfer mechanisms of direct bond interaction, shear connection, and direct bearing shall be determined in accordance with this section. Use of the force transfer *mechanism* providing the largest *nominal strength* is permitted. Force transfer mechanisms shall not be superimposed.

The force transfer mechanism of direct bond interaction shall not be used for encased composite members.

#### 3a. Direct Bearing

Where load is transferred in an encased or filled composite member by direct bearing from internal bearing mechanisms, the *available bearing strength* of the concrete for the *limit state of concrete crushing* shall be determined as follows:

$$R_n = 1.7f'_c A_1 \quad (I5-3)$$

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

where

$$A_1 = \text{loaded area of concrete, in.}^2 \text{ (mm}^2\text{)}$$

**User Note:** An example of force transfer via an internal bearing mechanism is the use of internal steel plates within a filled composite member.

#### 3b. Shear Connection

Where load is transferred in an encased or filled composite member by shear connection, the available shear strength of steel headed stud or steel channel anchors shall be determined as follows:

$$R_n = \Sigma Q_{cv} \quad (I5-4)$$

where

$\Sigma Q_{cv}$  = sum of *available shear strengths* of steel headed stud or steel channel anchors, determined in accordance with Section I7.3, placed within the *load introduction length* as defined in Section I5.4, kips (N).

#### 3c. Direct Bond Interaction

Where load is transferred in a filled composite member by direct bond interaction, the *available bond strength* between the steel and concrete shall be determined as follows:

$$\phi = 0.45 \text{ (LRFD)} \quad \Omega = 3.33 \text{ (ASD)}$$

For rectangular steel sections filled with concrete:

$$R_n = b^2 C_{in} F_{in} \quad (15-5)$$

For round steel sections filled with concrete:

$$R_n = 0.25\pi D^2 C_{in} F_{in} \quad (15-6)$$

where

$C_{in} = 2$  if the filled composite member extends to one side  
of the point of load transfer

$= 4$  if the filled composite member extends either side  
of the point of load transfer

$R_n$  = nominal bond strength, kips (N)

$F_{in}$  = nominal bond stress = 0.06 ksi (0.40 MPa)

$b$  = width of rectangular steel section along face transfer-  
ring load, in. (mm)

$D$  = diameter of round steel section, in. (mm)

## 4. Detailing Requirements

### 4a. Encased Composite Members

Steel anchors utilized to transfer longitudinal shear shall be distributed within the *load introduction length*, which shall not exceed a distance of two times the minimum transverse dimension of the encased composite member above and below the load transfer region. Anchors utilized to transfer longitudinal shear shall be placed on at least two faces of the steel shape in a generally symmetric configuration about the steel shape axes.

Steel anchor spacing, both within and outside of the load introduction length, shall conform to I7.3e.

### 4b. Filled Composite Members

Where required, steel anchors transferring the required longitudinal shear force shall be distributed within the *load introduction length*, which shall not exceed a distance of two times the minimum transverse dimension of a rectangular steel member or two times the diameter of a round steel member both above and below the load transfer region.

Steel anchor spacing within the load introduction length shall conform to Section I7.3e.

## 16. COMPOSITE DIAPHRAGMS AND COLLECTOR BEAMS

*Composite* slab diaphragms and collector *beams* shall be designed and detailed to transfer loads between the diaphragm, the diaphragm's boundary members and collector elements, and elements of the lateral-load-resisting-system.

**User Note:** Design guidelines for composite diaphragms and collector beams can be found in the commentary.

## 17. STEEL ANCHORS

### 1. General

The diameter of a steel headed stud anchor shall not be greater than 2.5 times the thickness of the base metal to which it is welded, unless it is welded to a flange directly over a web.

Section I7.2 applies for a *composite* flexural member where steel anchors are embedded in a solid concrete slab or in a slab cast on *formed steel* deck. Section I7.3 applies for all other cases.

### 2. Steel Anchors in Composite Beams

The length of steel headed stud anchors shall not be less than four stud diameters from the base of the steel headed stud anchor to the top of the stud head after installation.

#### 2a. Strength of Steel Headed Stud Anchors

The *nominal shear strength* of one steel headed stud anchor embedded in a solid concrete slab or in a composite slab with decking shall be determined as follows:

$$Q_n = 0.5 A_{sc} \sqrt{f'_c E_c} \leq R_g R_p A_{sc} F_u \quad (I7-1)$$

where

- $A_{sc}$  = cross-sectional area of steel headed stud anchor, in.<sup>2</sup> (mm<sup>2</sup>)  
 $E_c$  = modulus of elasticity of concrete =  $w_c^{1.5} \sqrt{f'_c}$ , ksi  
 (0.043  $w_c^{1.5} \sqrt{f'_c}$ , MPa)  
 $F_u$  = *specified minimum tensile strength* of a steel headed stud anchor, ksi (MPa)  
 $R_g$  = 1.0 for:

- 3821 (a) one steel headed stud anchor welded in a steel deck rib  
 3822 with the deck oriented perpendicular to the steel shape;  
 3823 (b) any number of steel headed stud anchors welded in a  
 3824 row directly to the steel shape;  
 3825 (c) any number of steel headed stud anchors welded in a  
 3826 row through steel deck with the deck oriented parallel to  
 3827 the steel shape and the ratio of the *average rib width* to  
 3828 rib depth  $\geq 1.5$   
 3829 = 0.85 for:  
 3830 (a) two steel headed stud anchors welded in a steel deck rib  
 3831 with the deck oriented perpendicular to the steel shape;  
 3832 (b) one steel headed stud anchor welded through steel deck  
 3833 with the deck oriented parallel to the steel shape and the  
 3834 ratio of the average rib width to rib depth  $< 1.5$   
 3835 = 0.7 for three or more steel headed stud anchors welded in a  
 3836 steel deck rib with the deck oriented perpendicular to the  
 3837 steel shape  
 3838  $R_p = 0.75$  for:  
 3839 (a) steel headed stud anchors welded directly to the steel  
 3840 shape;  
 3841 (b) steel headed stud anchors welded in a composite slab  
 3842 with the deck oriented perpendicular to the beam and  
 3843  $e_{mid-h_t} \geq 2$  in. (50 mm);  
 3844 (c) steel headed stud anchors welded through steel deck, or  
 3845 steel sheet used as *girder filler* material, and embedded  
 3846 in a composite slab with the deck oriented parallel to the  
 3847 beam  
 3848 = 0.6 for steel headed stud anchors welded in a composite  
 3849 slab with deck oriented *perpendicular* to the beam and  $e_{mid-}$   
 3850  $h_t < 2$  in. (50 mm)  
 3851  $e_{mid-h_t}$  = distance from the edge of steel headed stud anchor shank to  
 3852 the steel deck web, measured at mid-height of the deck rib,  
 3853 and in the *load* bearing direction of the steel headed stud  
 3854 anchor (in other words, in the direction of maximum mo-  
 3855 ment for a simply supported beam), in. (mm)  
 3856

**User Note:** The table below presents values for  $R_g$  and  $R_p$  for several cases. Capacities for steel headed stud anchors can be found in the Manual.

Condition	$R_g$	$R_p$
No decking	1.0	0.75
Decking oriented parallel to the steel shape		
$\frac{w_r}{h_r} \geq 1.5$	1.0	0.75
$\frac{w_r}{h_r} < 1.5$	0.85**	0.75
Decking oriented perpendicular		

to the steel shape		
Number of steel headed stud anchors occupying the same decking rib	1.0	0.6 <sup>+</sup>
1	0.85	0.6 <sup>+</sup>
2	0.7	0.6 <sup>+</sup>
3 or more		

$h_r$  = nominal rib height, in. (mm)

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

\*\* for a single steel headed stud anchor

<sup>+</sup> this value may be increased to 0.75 when  $e_{mid-hr} \geq 2$  in. (51 mm)

## 2b. Strength of Steel Channel Anchors

The nominal shear strength of one hot-rolled channel anchor embedded in a solid concrete slab shall be determined as follows:

$$Q_n = 0.3(t_f + 0.5t_w)l_c\sqrt{f'_cE_c} \quad (I7-2)$$

where

$t_f$  = flange thickness of channel anchor, in. (mm)

$t_w$  = web thickness of channel anchor, in. (mm)

$l_c$  = length of channel anchor, in. (mm)

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

## 2c. Required Number of Steel Anchors

The number of anchors required between the section of maximum bending moment, positive or negative, and the adjacent section of zero moment shall be equal to the *horizontal shear* as determined in Sections I3.2d(1) and I3.2d(2) divided by the nominal shear strength of one steel anchor as determined from Section I7.2a or Section I7.2b. The number of steel anchors required between any concentrated load and the nearest point of zero moment shall be sufficient to develop the maximum moment required at the concentrated load point.

## 2d. Detailing Requirements

Steel anchors required on each side of the point of maximum bending moment, positive or negative, shall be distributed uniformly between that point and the adjacent points of zero moment, unless specified

otherwise on the contract documents.

Steel anchors shall have at least 1 in. (25 mm) of lateral concrete cover in the direction perpendicular to the shear force, except for anchors installed in the ribs of formed steel decks. The minimum distance from the center of an anchor to a free edge in the direction of the shear force shall be 8 in. (203 mm) if normal weight concrete is used and 10 in. (250 mm) if lightweight concrete is used. The provisions of ACI 318 Appendix D are permitted to be used in lieu of these values.

The minimum center-to-center spacing of steel headed stud anchors shall be six diameters along the longitudinal axis of the supporting composite beam and four diameters transverse to the longitudinal axis of the supporting composite beam, except that within the ribs of formed steel decks oriented perpendicular to the steel beam the minimum center-to-center spacing shall be four diameters in any direction. The maximum center-to-center spacing of steel anchors shall not exceed eight times the total slab thickness or 36 in (900 mm).

### 3. Steel Anchors in Composite Components

This section shall apply to the design of steel headed stud anchors in *composite components*.

For normal weight concrete: Steel headed stud anchors subjected to shear only shall not be less than five stud diameters in length from the base of the steel headed stud to the top of the stud head after installation. Steel headed stud anchors subjected to tension or interaction of shear and tension shall not be less than eight stud diameters in length from the base of the stud to the top of the stud head after installation,

For lightweight concrete: Steel headed stud anchors subjected to shear only shall not be less than seven stud diameters in length from the base of the steel headed stud to the top of the stud head after installation. Steel headed stud anchors subjected to tension shall not be less than ten stud diameters in length from the base of the stud to the top of the stud head after installation. The nominal strength of steel headed stud anchors subjected to interaction of shear and tension for lightweight concrete shall be determined as stipulated by the *applicable building code* or ACI 318 Appendix D.

Steel headed stud anchors subjected to tension or interaction of shear and tension shall have a diameter of the head greater than or equal to 1.6 times the diameter of the shank.

**User Note:** The steel anchor strength provisions in this section are applicable to composite components such as composite columns (including load transfer regions), concrete-encased and filled composite beams, composite beam-columns, coupling beams, and composite walls. Section I7.2 specifies the strength of steel anchors embedded in

a solid concrete slab or in a concrete slab with formed steel deck in a composite beam.

### 3a. Shear Strength of Steel Headed Stud Anchors in Composite Components

Where the distance from the center of an anchor to a free edge of concrete in the direction perpendicular to the height of the steel headed stud anchor is greater than or equal to 1.5 times the height of the steel headed stud anchor measured to the top of the stud head, the nominal shear strength of one steel headed stud anchor shall be determined as follows:

$$Q_{nv} = A_{sc} F_u \quad (17-3)$$

$$\phi_v = 0.65 \text{ (LRFD)} \quad \Omega_v = 2.31 \text{ (ASD)}$$

where

$Q_{nv}$  = nominal shear strength of steel headed stud anchor, kips (N) (mm<sup>2</sup>)

$A_{sc}$  = cross-sectional area of steel headed stud anchor, in.<sup>2</sup> (mm<sup>2</sup>)

$F_u$  = specified minimum tensile strength of a steel headed stud anchor, ksi (MPa)

Where the distance from the center of an anchor to a free edge of concrete in the direction perpendicular to the height of the steel headed stud anchor is less than 1.5 times the height of the steel headed stud anchor measured to the top of the stud head, the nominal shear strength of one steel headed stud anchor shall be determined in accordance with ACI 318 Appendix D.

### 3b. Tensile Strength of Steel Headed Stud Anchors in Composite Components

Where the distance from the center of an anchor to a free edge of concrete in the direction perpendicular to the height of the steel headed stud anchor is greater than or equal to 1.5 times the height of the steel headed stud anchor measured to the top of the stud head, and where the center-to-center spacing of steel headed stud anchors is greater than or equal to three times the height of the steel headed stud anchor measured to the top of the stud head, the nominal tensile strength of one steel headed stud anchor shall be determined as follows:

$$Q_{nt} = A_{sc} F_u \quad (17-4)$$

$$\phi_s = 0.75 \text{ (LRFD)} \quad \Omega_s = 2.00 \text{ (ASD)}$$

where

$Q_{nt}$  = nominal tensile strength of steel headed stud anchor, kips (N) (mm<sup>2</sup>)



Where the distance from the center of an anchor to a free edge of concrete in the direction perpendicular to the height of the steel headed stud anchor is less than 1.5 times the height of the steel headed stud anchor measured to the top of the stud head, or where the center-to-center spacing of steel headed stud anchors is less than three times the height of the steel headed stud anchor measured to the top of the stud head, the nominal tensile strength of one steel headed stud anchor shall be determined by one of the following:

- 1) Where anchor reinforcement is developed in accordance with Chapter 12 of ACI 318 on opposite sides of the *concrete breakout surface* for the steel headed stud anchor, the minimum of the steel nominal tensile strength from Eq. I7-4 and the nominal strength of the anchor reinforcement shall be used for the nominal tensile strength,  $Q_{nt}$ , of the steel headed stud anchor.
- 2) As stipulated by the applicable building code or ACI 318 Appendix D.

**User Note:** Supplemental confining reinforcement is recommended around the anchors for steel headed stud anchors subjected to tension or interaction of shear and tension to avoid edge effects or effects from closely spaced anchors. See the Commentary and ACI 318 Section D5.2.9 for guidelines.

### 3c. Strength of Steel Headed Stud Anchors for Interaction of Shear and Tension in Composite Components

Where the distance from the center of an anchor to a free edge of concrete in the direction perpendicular to the height of the steel headed stud anchor is greater than or equal to 1.5 times the height of the steel headed stud anchor measured to the top of the stud head, and where the center-to-center spacing of steel headed stud anchors is greater than or equal to three times the height of the steel headed stud anchor measured to the top of the stud head, the nominal strength for interaction of shear and tension of one steel headed stud anchor shall be determined as follows:

$$\left[ \left( \frac{Q_{rt}}{Q_{ct}} \right)^{5/3} + \left( \frac{Q_{rv}}{Q_{cv}} \right)^{5/3} \right] \leq 1.0 \quad (I7-5)$$

$Q_{rt}$  = required tensile strength, kips (N)  
 $Q_{ct}$  = available tensile strength, kips (N)  
 $Q_{rv}$  = required shear strength, kips (N)  
 $Q_{cv}$  = available shear strength, kips (N)

**For design in accordance with Section B3.3 (LRFD):**

$Q_{rt}$  = required tensile strength using LRFD load combinations, kips (N)

$Q_{ct} = \phi_t Q_{nt}$  = design tensile strength, determined in accordance with Section I7.3b, kips (N)

$Q_{rv}$  = required shear strength using *LRFD* load combinations, kips (N)

$Q_{cv} = \phi_v Q_{nv}$  = design shear strength, determined in accordance with Section I7.3a, kips (N)

$\phi_t$  = resistance factor for tension = 0.75

$\phi_v$  = resistance factor for flexure = 0.65

**For design in accordance with Section B3.4 (ASD):**

$Q_{rt}$  = required tensile strength using ASD load combinations, kips (N)

$Q_{ct} = Q_{nt}/\Omega_t$  = allowable tensile strength, determined in accordance with Section I7.3b, kips (N)

$Q_{rv}$  = required shear strength using ASD load combinations, kips (N)

$Q_{cv} = Q_{nv}/\Omega_v$  = allowable shear strength, determined in accordance with Section I7.3a, kips (N)

$\Omega_t$  = safety factor for tension = 2.00

$\Omega_v$  = safety factor for flexure = 2.31

Where the distance from the center of an anchor to a free edge of concrete in the direction perpendicular to the height of the steel headed stud anchor is less than 1.5 times the height of the steel headed stud anchor measured to the top of the stud head, or where the center-to-center spacing of steel headed stud anchors is less than three times the height of the steel headed stud anchor measured to the top of the stud head, the nominal strength for interaction of shear and tension of one steel headed stud anchor shall be determined by one of the following:

- 1) Where anchor reinforcement is developed in accordance with Chapter 12 of ACI 318 on opposite sides of the *concrete breakout surface* for the steel headed stud anchor, the minimum of the steel nominal tensile strength from Eq. I7-4 and the nominal strength of the anchor reinforcement shall be used for the nominal tensile strength,  $Q_{nt}$ , of the steel headed stud anchor for use in Eq. I7-5.
- 2) As stipulated by the applicable building code or ACI 318 Appendix D.

**3d. Shear Strength of Steel Channel Anchors in Composite Components**

The provisions of I7.2b apply with the resistance factor and safety factor as specified below.

$$\phi_s = 0.75 \text{ (LRFD)} \quad \Omega_s = 2.00 \text{ (ASD)}$$

**3e. Detailing Requirements in Composite Components**

Steel anchors shall have at least 1 in. (25 mm) of lateral clear concrete cover. The minimum center-to-center spacing of steel headed stud anchors shall be four diameters in any direction. The maximum center-to-center spacing of steel headed stud anchors shall not exceed 24 times the shank diameter. The maximum center-to-center spacing of steel of channel anchors shall be 24 in. (600 mm).

**18. SPECIAL CASES**

When *composite* construction does not conform to the requirements of Section I1 through Section I7, the strength of *steel anchors* and details of construction shall be established by testing.

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## CHAPTER J

### DESIGN OF CONNECTIONS

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

The chapter is organized as follows:

- J1. General Provisions
- J2. Welds
- J3. Bolts and Threaded Parts
- J4. Affected Elements of Members and Connecting Elements
- J5. Fillers
- J6. Splices
- J7. Bearing Strength
- J8. Column Bases and Bearing on Concrete
- J9. Anchor Rods and Embedments
- J10. Flanges and Webs with Concentrated Forces

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

- Chapter K. Design of HSS and Box Member Connections
- Appendix 3. Design for Fatigue

#### J1. GENERAL PROVISIONS

##### 1. Design Basis

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

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

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

##### 2. Simple Connections

*Simple connections* of *beams*, girders and trusses shall be designed as

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

### 3. Moment Connections

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

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

### 4. Compression Members with Bearing Joints

Compression members with bearing plates shall meet the following requirements:

- (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 their required strength shall be the lesser of:
  - (i) An axial tensile force of 50% of the required compressive strength of the member; or
  - (ii) The moment and shear resulting from a transverse *load* equal to 2% of the required compressive strength of the member. The transverse load shall be applied at the location of the splice exclusive of other loads that act on the member. The member shall be taken as pinned for the determination of the shears and moments at the splice.

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

### 5. Splices in Heavy Sections

When tensile forces due to applied tension or flexure are to be transmitted through splices in heavy sections, as defined in Section A3.1c and A3.1d, by complete-joint-penetration groove (CJP) welds, material notch-toughness requirements as given in Section A3.1c and

A3.1d, weld access hole details as given in Section J1.6, filler metal requirements as given in Section J2.6 and thermal cut surface preparation and inspection requirements as given in Section M2.2 shall apply. The foregoing provision is not applicable to splices of elements of built-up shapes that are welded prior to assembling the shape.

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

## 6. Weld Access Holes

All weld access holes required to facilitate welding operations shall be detailed to provide room for weld backing as needed. The access hole 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, nor less than  $1\frac{1}{2}$  in. (38 mm). The access hole shall have a height not less than the thickness of the material with the access hole, nor less than  $\frac{3}{4}$  in. (19 mm) nor does it need to exceed 2 in. (50 mm).

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, 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  $\frac{3}{8}$  in. (10 mm).

In built-up shapes with fillet or *partial-joint-penetration groove welds* that join the web-to-flange, 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 A3.1c and A3.1d, the *thermally cut* surfaces of 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 is formed by predrilled or sawed holes, that portion of the access hole need not be ground. Weld access holes in other shapes need not be ground nor inspected by dye penetrant or magnetic particle methods.

## 7. Placement of Welds and Bolts

Groups of welds or bolts at the ends of any member which transmit axial force into that member shall be sized so that the center of gravity

of the group coincides with the center of gravity of the member, unless provision is made for the eccentricity. The foregoing provision is not applicable to end connections of single angle, double angle and similar members.

## 8. Bolts in Combination with Welds

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

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

## 9. High-Strength Bolts in Combination with Rivets

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

## 10. Limitations on Bolted and Welded Connections

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

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

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

## J2. WELDS

All provisions of AWS D1.1, apply under this Specification, with the exception that the provisions of the listed AISC Specification Sections

apply under this Specification in lieu of the cited AWS provisions as follows:

- (1) Section J1.6 in lieu of AWS D1.1 Section 5.17.1
- (2) Section J2.2a in lieu of AWS D1.1 Section 2.3.2
- (3) Table J2.2 in lieu of AWS D1.1 Table 2.1
- (4) Table J2.5 in lieu of AWS D1.1 Table 2.3
- (5) Appendix 3, Table A-3.1 in lieu of AWS D1.1 Table 2.4
- (6) Section B3.10 and Appendix 3 in lieu of AWS D1.1 Section 2, Part C
- (7) Section M2.2 in lieu of AWS D1.1 Sections 5.15.4.3 and 5.15.4.4

## 1. Groove Welds

### 1a. Effective Area

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

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

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

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

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

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

### 1b. Limitations

The minimum effective throat of a partial-joint-penetration groove weld shall not be less than the size required to transmit calculated



forces nor the size shown in Table J2.3. Minimum weld size is determined by the thinner of the two parts joined.

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

Welding Process	Welding Position F (flat), H (horizontal), V (vertical), OH (overhead)	Groove Type (AWS D1.1, Figure 3.3)	Effective Throat
Shielded metal arc (SMAW)	All	J or U groove  60° V	depth of groove
Gas metal arc (GMAW) Flux cored arc (FCAW)	All		
Submerged arc (SAW)	F	J or U groove  60° bevel or V	
Gas metal arc (GMAW) Flux cored arc (FCAW)	F, H	45° bevel	depth of groove
Shielded metal arc (SMAW)	All	45° bevel	depth of groove minus 1/8 in. (3 mm)
Gas metal arc (GMAW) Flux cored arc (FCAW)	V, OH	45° bevel	depth of groove minus 1/8 in. (3 mm)

**TABLE J2.2**  
**Effective Weld Sizes of**  
**Flare Groove Welds**

Welding Process	Flare Bevel Groove <sup>[a]</sup>	Flare V Groove
GMAW and FCAW-G	5/8 <i>R</i>	3/4 <i>R</i>
SMAW and FCAW-S	5/16 <i>R</i>	5/8 <i>R</i>
SAW	5/16 <i>R</i>	1/2 <i>R</i>
<sup>[a]</sup> For flare bevel groove with <i>R</i> < 3/8 in. (10 mm) use only reinforcing fillet weld on filled flush joint. General note: <i>R</i> = radius of joint surface (can be assumed to be 2 <i>t</i> for HSS), in. (mm)		

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

Material Thickness of Thinner Part Joined, in. (mm)	Minimum Effective Throat Thickness, <sup>[a]</sup> in. (mm)
To 1/4 (6) inclusive	1/8 (3)
Over 1/4 (6) to 1/2 (13)	3/16 (5)
Over 1/2 (13) to 3/4 (19)	1/4 (6)
Over 3/4 (19) to 1 1/2 (38)	5/16 (8)
Over 1 1/2 (38) to 2 1/4 (57)	3/8 (10)
Over 2 1/4 (57) to 6 (150)	1/2 (13)

Over 6 (150)	5/8 (16)
[a] See Table J2.1.	

## 2. Fillet Welds

### 2a. Effective Area

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

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

### 2b. Limitations

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

**TABLE J2.4**  
**Minimum Size of Fillet Welds**

Material Thickness of Thinner Part Joined, in. (mm)	Minimum Size of Fillet Weld, [a] in. (mm)
To 1/4 (6) inclusive	1/8 (3)
Over 1/4 (6) to 1/2 (13)	3/16 (5)
Over 1/2 (13) to 3/4 (19)	1/4 (6)
Over 3/4 (19)	5/16 (8)
[a] Leg dimension of fillet welds. Single pass welds must be used. Note: See Section J2.2b for maximum size of fillet welds.	

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

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

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

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,  $\beta$ , determined as follows:

$$\beta = 1.2 - 0.002(l/w) \leq 1.0 \quad (J2-1)$$

where

$l$  = actual length of end-loaded weld, in. (mm)

$w$  = size of weld leg, in. (mm)

When the length of the weld exceeds 300 times the leg size,  $w$ , the effective length shall be taken as  $180w$ .

*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 1½ in. (38 mm).

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

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

- (1) For overlapping elements of members in which one connected part extends beyond an edge of another connected part that is subject to calculated tensile stress, fillet welds shall terminate not less than the size of the weld from that edge.

- (2) For *connections* where flexibility of the outstanding elements is required, when *end returns* are used, the length of the return shall not exceed four times the nominal size of the weld nor half the width of the part.
- (3) Fillet welds joining *transverse stiffeners* to *plate girder* webs  $\frac{3}{4}$  in. (19 mm) thick or less shall end not less than four times nor more than six times the thickness of the web from the web toe of the web-to-flange welds, except where the ends of *stiffeners* are welded to the flange.
- (4) Fillet welds that occur on opposite sides of a common plane shall be interrupted at the corner common to both welds.

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

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

### 3. Plug and Slot Welds

#### 3a. Effective Area

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

#### 3b. Limitations

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

The diameter of the holes for a plug weld shall not be less than the thickness of the part containing it plus  $\frac{5}{16}$  in. (8 mm), rounded to the next larger odd  $\frac{1}{16}$  in. (even mm), nor greater than the minimum diameter plus  $\frac{1}{8}$  in. (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  $\frac{5}{16}$  in. (8 mm) rounded to the next larger odd  $\frac{1}{16}$  in. (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  $\frac{5}{8}$  in. (16 mm) or less in thickness shall be equal to the thickness of the material. In material over  $\frac{5}{8}$  in. (16 mm) thick, the thickness of the weld shall be at least one-half the thickness of the material but not less than  $\frac{5}{8}$  in. (16 mm).

#### 4. Strength

The *design strength*,  $\phi R_n$  and the *allowable strength*,  $R_n/\Omega$ , of welded joints shall be the lower value of the base material strength determined according to the *limit states* of *tensile rupture* and *shear rupture* and the weld metal strength determined according to the limit state of *yielding* as follows:

For the base metal

$$R_n = F_{nBM} A_{BM} \quad (J2-2)$$

For the weld metal

$$R_n = F_{nw} A_{we} \quad (J2-3)$$

where

$F_{nBM}$  = nominal strength per unit area of the base metal, ksi (MPa)

$F_{nw}$  = nominal strength per unit area of the weld metal, ksi (MPa)

$A_{BM}$  = cross-sectional area of the base metal, in.<sup>2</sup> (mm<sup>2</sup>)

$A_{we}$  = effective area of the weld, in.<sup>2</sup> (mm<sup>2</sup>)

The values of  $\phi$ ,  $\Omega$ ,  $F_{nBM}$  and  $F_{nw}$  and limitations thereon are given in Table J2.5.

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

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

Tension or compression Parallel to weld axis	Tension or compression in parts joined parallel to a weld need not be considered in design of welds joining the parts.				Filler metal with a strength level equal to or less than matching filler metal is permitted.
Shear	Base	Governed by J4			
	Weld	$\phi = 0.75$ $\Omega = 2.00$	$0.60 F_{EXX}$	See J2.1a	
FILLET WELDS INCLUDING FILLETS IN HOLES AND SLOTS AND SKEWED T –JOINTS					
Shear	Base	Governed by J4			Filler metal with a strength level equal to or less than matching filler metal is permitted.
	Weld	$\phi = 0.75$ $\Omega = 2.00$	$0.60F_{EXX}^{[d]}$	See J2.2a	
Tension or compression Parallel to weld axis	Tension or compression in parts joined parallel to a weld need not be considered in design of welds joining the parts.				
PLUG AND SLOT WELDS					
Shear Parallel to faying surface on the effective area	Base	Governed by J4			Filler metal with a strength level equal to or-less than matching filler metal is permitted
	Weld	$\phi = 0.75$ $\Omega = 2.00$	$0.60 F_{EXX}$	J2.3a	
<sup>[a]</sup> For matching weld metal see AWS D1.1, Section 3.3.					
<sup>[b]</sup> Filler metal with a strength level one strength level greater than matching is permitted.					
<sup>[c]</sup> 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.					
<sup>[d]</sup> Alternatively, the provisions of Section J2.4(a) are permitted provided the deformation compatibility of the various weld elements is considered. Sections J2.4(b) and (c) are special applications of Section J2.4(a) that provide for deformation compatibility.					

Alternatively, for *fillet welds* loaded in-plane the *available* strength is permitted to be determined as follows:

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

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

$$R_n = F_{nw} A_{we} \qquad (J2-4)$$

where

$$F_{nw} = 0.60 F_{EXX} (1.0 + 0.50 \sin^{1.5} \theta) \qquad (J2-5)$$

and

$F_{EXX}$  = electrode classification strength, ksi (MPa)

$\theta$  = angle of loading measured from the weld longitudinal axis, degrees

**User Note:** A linear weld group is one in which all elements are in

a line or are parallel.

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

$$R_{nx} = \sum F_{nwix} A_{wei} \quad R_{ny} = \sum F_{nwi y} A_{wei} \quad (J2-6)$$

$$M_n = \sum [F_{nwi y} A_{wei} (x_i) - F_{nwix} A_{wei} (y_i)] \quad (J2-7)$$

where

$A_{wei}$  = effective area of weld throat of any  $i$ th weld element, in.<sup>2</sup>  
(mm<sup>2</sup>)

$$F_{nwi} = 0.60 F_{EXX} (1.0 + 0.50 \sin^{1.5} \theta_i) f(p_i) \quad (J2-8)$$

$$f(p_i) = [p_i (1.9 - 0.9 p_i)]^{0.3} \quad (J2-9)$$

$F_{nwi}$  = nominal *stress* in any  $i$ th weld element, ksi (MPa)

$F_{nwix}$  =  $x$  component of stress,  $F_{nwi}$ , ksi (MPa)

$F_{nwi y}$  =  $y$  component of stress,  $F_{nwi}$ , ksi (MPa)

$p_i$  =  $\Delta_i / \Delta_{mi}$ , ratio of element  $i$  deformation to its deformation  
at maximum stress

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

$r_i$  = distance from instantaneous center of rotation to  $i^{th}$  weld  
element, in. (mm)

$x_i$  =  $x$  component of  $r_i$

$y_i$  =  $y$  component of  $r_i$

$\Delta_i$  = deformation of weld elements at intermediate stress levels,  
linearly proportioned to the critical deformation based  
on distance from the instantaneous center of rotation,  
 $r_i$ , in. (mm)

$$= r_i \Delta_{ui} / r_{crit}$$

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

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

$\theta_i$  = angle of loading measured from the longitudinal axis of  $i^{th}$   
weld element, degrees

(c) For fillet weld groups concentrically loaded and consisting of  
elements with a uniform leg size that are oriented both longitudi-  
nally and transversely to the direction of applied *load*, the com-  
bined strength,  $R_n$ , of the fillet weld group shall be determined as  
the greater of

$$(i) R_n = R_{nwl} + R_{nwt} \quad (J2-10a)$$

or

$$(ii) R_n = 0.85 R_{nwl} + 1.5 R_{nwt} \quad (J2-10b)$$



where

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

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

## 5. Combination of Welds

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

## 6. Filler Metal Requirements

The choice of filler metal 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.

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

Base Metal		Matching Filler Metal
A36 $\leq 3/4$ in. thick		60 & 70 ksi Electrodes
A36 $> 3/4$ in.	A572 (Gr. 50 & 55)	SMAW: E7015, E7016, E7018, E7028 Other processes: 70 ksi electrodes
A588*	A913 (Gr. 50)	
A1011	A992	
	A1018	
A913	(Gr. 60 & 65)	80 ksi electrodes
<p>* For corrosion resistance and color similar to the base see AWS D1.1, Sect. 3.7.3</p> <p>Notes:</p> <p>1. Electrodes shall meet the requirements of AWS A5.1, A5.5, A5.17, A5.18, A5.20, A5.23, A5.28 or A5.29.</p> <p>In joints with base metals of different strengths use either a filler metal that matches the higher strength base metal or a filler metal that matches the lower strength and produces a low hydrogen deposit.</p>		

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

- (1) Complete-joint-penetration groove welded T and corner joints with steel backing left in place, subject to tension normal to the effective area, unless the joints are designed using the *nominal*

*strength and resistance factor or safety factor* as applicable for a partial-joint-penetration groove weld.

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

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

## 7. Mixed Weld Metal

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

## J3. BOLTS AND THREADED PARTS

### 1. High-Strength Bolts

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

Group A – ASTM A325, A325M, F1582, A354 Grade BC and A449

Group B – ASTM A490, A490M, F2280 and A354 Grade BD

When assembled, all *joint* surfaces, including those adjacent to the washers, shall be free of scale, except tight *mill scale*. All high-strength bolts shall be tightened to a bolt tension not less than that given in Table J3.1 or J3.1M, except as noted below. Except as permitted below, installation shall be assured by any of the following methods: turn-of-nut method, a direct-tension-indicator, twist-off-type tensions-control bolt, calibrated wrench or alternative design bolt.

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

- (a) *bearing-type connections*, or
- (b) tension or combined shear and tension applications, for Group A bolts only, where loosening or *fatigue* due to vibration or *load* fluctuations are not design considerations.

The snug-tight condition is defined as the tightness attained by either a few impacts of an impact wrench or the full effort of a worker with an ordinary spud wrench that brings the connected plies into firm

contact. Bolts to be tightened to a condition other than snug tight shall be clearly identified on the design drawings.

**User Note:** Bolts tightened in excess of snug tight are permissible where snug tight is specified.

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

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

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

When bolt requirements cannot be provided within the RCSC Specification limitations because of requirements for lengths exceeding 12 diameters or diameters exceeding  $1\frac{1}{2}$  in. (38 mm), bolts or threaded rods conforming to Group A or Group B materials are permitted to be used in accordance with the provisions for threaded rods in Table J3.2.

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

## 2. Size and Use of Holes

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

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

**TABLE J3.1**  
**Minimum Bolt Pretension, kips \***

Bolt Size, in.	Group A - A325, F1852 Bolts	Group B - A490, F2280 Bolts
½	12	15
5/8	19	24
¾	28	35
7/8	39	49
1	51	64
1 1/8	56	80
1 ¼	71	102
1 3/8	85	121
1 ½	103	148

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

**TABLE J3.1M**  
**Minimum Bolt Pretension, kN \***

Bolt Size, mm	Group A - A325M Bolts	Group B - 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.

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  $\frac{5}{16}$  in. (8 mm) thick and shall be of structural grade material, but need not be hardened. If hardened washers are required for use of high-strength bolts, the hardened washers shall be placed over the outer surface of the plate washer or bar.

### 3. Minimum Spacing

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

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

Description of Fasteners	Nominal Tensile Strength, $F_{nt}$ ksi (MPa)	Nominal Shear Strength in Bearing-Type Connections, $F_{nv}$ ksi (MPa)	
		Fastener Pattern Length <sup>[a]</sup>	
		≤ 38 in. (965 mm)	> 38 in. (965 mm)
A307 bolts	45 (310) <sup>[b][c]</sup>	27 (188) <sup>[c]</sup> [d]	23 (159) <sup>[c][d]</sup> [e]
Group A (A325 type) bolts, when threads are not excluded from shear planes	90 (620) <sup>[e]</sup>	54 (372)	45(310) <sup>[f]</sup>
Group A (A325 type) bolts, when threads are excluded from shear planes	90 (620) <sup>[e]</sup>	68 (457)	56 (388) <sup>[f]</sup>
Group B (A490 type) bolts, when threads are not excluded from shear planes	113 (780) <sup>[e]</sup>	68 (457)	56 (388) <sup>[f]</sup>
Group B (A490 type) bolts, when threads are excluded from shear planes	113 (780) <sup>[e]</sup>	84 (579)	70 (483) <sup>[f]</sup>
Threaded parts meeting the requirements of Section A3.4, when threads are not excluded from shear planes	$0.75 F_u$ <sup>[b]</sup>	$0.450 F_u$	$0.375 F_u$ <sup>[f]</sup>
Threaded parts meeting the requirements of Section A3.4, when threads are excluded from shear planes	$0.75 F_u$ <sup>[b]</sup>	$0.563 F_u$	$0.469 F_u$ <sup>[f]</sup>
<sup>[a]</sup> Fastener pattern length is defined as the distance measured parallel to the line of force from the centerline of the end bolts for lap splices. For butt splices, the distance is measured from the centerline to the end bolt to the centerline of the bolt nearest the connection center. <sup>[b]</sup> Subject to the requirements of Appendix 3. <sup>[c]</sup> For A307 bolts the tabulated values shall be reduced by 1% for each 1/16 in. (2 mm) over 5 diameters of length in the grip. <sup>[d]</sup> Threads permitted in shear planes. <sup>[e]</sup> For <i>high-strength bolts</i> subject to tensile fatigue loading, see Appendix 3. <sup>[f]</sup> Applicable only to end-loaded connections.			

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**User Note:** ASTM F1554 anchor rods may be furnished in accordance to product specifications with a body diameter less than the nominal diameter. Load effects such as bending and elongation should be calculated based on minimum diameters permitted by the product specification. See ASTM F1554 and Manual Table 2-5.

#### 4. Minimum Edge Distance

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

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

#### 5. Maximum Spacing and Edge Distance

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

**TABLE J3.3**  
Nominal Hole Dimensions, in.

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

**TABLE J3.3M**  
Nominal Hole Dimensions, mm

Bolt Diameter	Hole Dimensions			
	Standard (Dia.)	Oversize (Dia.)	Short-Slot (Width x Length)	Long-Slot (Width x Length)
M16	18	20	18 x 22	18 x 40
M20	22	24	22 x 26	22 x 50

M22	24	28	24 x 30	24 x 55
M24	27 <sup>[a]</sup>	30	27 x 32	27 x 60
M27	30	35	30 x 37	30 x 67
M30	33	38	33 x 40	33 x 75
≥M36	$d + 3$	$d + 8$	$(d + 3) \times (d + 10)$	$(d + 3) \times 2.5d$
<sup>[a]</sup> Clearance provided allows the use of a 1 in. bolt if desirable.				

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

Bolt Diameter (in.)	Minimum Edge Distance (in.)
$\frac{1}{2}$	$\frac{3}{4}$
$\frac{5}{8}$	$\frac{7}{8}$
$\frac{3}{4}$	1
$\frac{7}{8}$	$1\frac{1}{8}$
1	$1\frac{1}{4}$
$1\frac{1}{8}$	$1\frac{1}{2}$
$1\frac{1}{4}$	$1\frac{5}{8}$
Over $1\frac{1}{4}$	$1\frac{1}{4} \times d$
<sup>[a]</sup> If necessary, lesser edge distances are permitted provided the appropriate provisions from Sections J3.10 and J4 are satisfied, but edge distances less than (1) bolt diameter are not permitted without approval from the engineer of record. <sup>[b]</sup> For oversized or slotted holes, see Table J3.5.	

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

Bolt Diameter (mm)	Minimum Edge Distance (mm)
16	22
20	26
22	28
24	30
27	34
30	38
36	46
Over 36	$1.25d$
<sup>[a]</sup> If necessary, lesser edge distances are permitted provided the appropriate provisions from Sections J3.10 and J4 are satisfied, but edge distances less than (1) bolt diameter are not permitted without approval from the engineer of record. <sup>[b]</sup> For oversized or slotted holes, see Table J3.5M.	

- (a) For painted members or unpainted members not subject to corrosion, the spacing shall not exceed 24 times the thickness of the thinner part or 12 in. (305 mm).
- (b) For unpainted members of *weathering steel* subject to atmospheric corrosion, the spacing shall not exceed 14 times the thickness of the thinner part or 7 in. (180 mm).

**User Note:** Dimensions in (a) and (b) do not apply to elements consisting of two shapes in continuous contact

## 6. Tension and Shear Strength of Bolts and Threaded Parts

The *design tension* or *shear strength*,  $\phi R_n$ , and the *allowable tension* or *shear strength*,  $R_n/\Omega$ , 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:

$$R_n = F_n A_b \quad (\text{J3-1})$$

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

Bolt shear strength,  $F_{nv}$ , for connections less than or equal to 38 in. (965 mm) in Table J3.2 may be used regardless of the connection length where the connection geometry, bolt properties, and connected material properties satisfy:

$$A_g \geq 0.53 A_s F_{ub}/F_y \text{ and} \quad (\text{J3-1b})$$

$$A_n \geq 0.53 A_s F_{ub}/F_u \quad (\text{J3-1c})$$

where

$A_b$  = nominal unthreaded body area of bolt or threaded part (for upset rods, see footnote d, Table J3.2), in.<sup>2</sup> (mm<sup>2</sup>)

$A_g$  = gross section of connected material, in.<sup>2</sup> (mm<sup>2</sup>)

$A_n$  = effective net area of connected material, in.<sup>2</sup> (mm<sup>2</sup>)

$A_s$  = total bolt area included in all shear planes. With threads in the shear plane,  $A_s$  is taken as 0.80 times the nominal area of the bolts, in.<sup>2</sup> (mm<sup>2</sup>)

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

$F_{ub}$  = nominal tensile strength of bolt or threaded material, ksi (MPa)

$F_u$  = specified minimum tensile strength of the connected material, ksi (MPa)

$F_y$  = specified minimum yield stress of the connected material, ksi (MPa)



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

## 7. Combined Tension and Shear in Bearing-Type Connections

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

$$R_n = F'_t A_b \quad (\text{J3-2})$$

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

where

$F'_t$  = nominal tensile stress modified to include the effects of shear stress, ksi (MPa)

$$F'_t = 1.3F_t - \frac{F_t}{\phi F_{nv}} f_{rv} \leq F_t \text{ (LRFD)} \quad (\text{J3-3a})$$

$$F'_t = 1.3F_t - \frac{\Omega F_t}{F_{nv}} f_{rv} \leq F_t \text{ (ASD)} \quad (\text{J3-3b})$$

$F_t$  = nominal tensile stress from Table J3.2, ksi (MPa)

$F_{nv}$  = nominal shear stress from Table J3.2, ksi (MPa)

$f_{rv}$  = required shear stress using LRFD or ASD load combinations, ksi (MPa)

The available shear stress of the *fastener* shall equal or exceed the required shear stress,  $f_{rv}$ .

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

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

Nominal Diameter of Fastener (in.)	Oversized Holes	Slotted Holes		
		Long Axis Perpendicular to Edge		Long Axis Parallel to Edge
		Short Slots	Long Slots <sup>[a]</sup>	
≤ 7/8	1/16	1/8	3/4d	0
1	1/8	1/8		
≥ 1 1/8	1/8	3/16		

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

TABLE J3.5M

Values of Edge Distance Increment  $C_2$ , mm

Nominal Diameter of Fastener (mm)	Oversized Holes	Slotted Holes		
		Long Axis Perpendicular to Edge		Long Axis Parallel to Edge
		Short Slots	Long Slots <sup>[a]</sup>	
≤ 22	2	3	0.75d	0
24	3	3		
≥ 27	3	5		
<sup>[a]</sup> When length of slot is less than maximum allowable (see Table J3.3M), C <sub>2</sub> is permitted to be reduced by one-half the difference between the maximum and actual slot lengths.				

**8. High-Strength Bolts in Slip-Critical Connections**

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

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

$$R_n = \mu D_u h_f T_b n_s \quad (\text{J3-4})$$

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

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

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

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

For long-slotted holes

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

where

$\mu$  = mean slip coefficient for Class A or B surfaces, as applicable, and determined as follows, or as established by tests:

- (1) For Class A surfaces (unpainted clean mill scale steel surfaces or surfaces with Class A coatings on blast-cleaned steel or hot-dipped galvanized and roughened surfaces)

$$\mu = 0.30$$

- (2) For Class B surfaces (unpainted blast-cleaned steel surfaces or surfaces with Class B coatings on blast-

cleaned steel)

$$\mu = 0.50$$

$D_u = 1.13$ ; 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.

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

$h_f$  = factor for fillers, determined as follows:

- (1) Where bolts have been added to distribute loads in the filler

$$h_f = 1.0$$

- (2) Where bolts have not been added to distribute the load in the filler:

- (i) For one filler between connected parts

$$h_f = 1.0$$

- (ii) For two or more fillers between connected parts

$$h_f = 0.85$$

$n_s$  = number of slip planes required to permit the connection to slip

## 9. Combined Tension and Shear in Slip-Critical Connections

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

$$k_{sc} = 1 - \frac{T_u}{D_u T_b n_b} \quad (\text{LRFD}) \quad (\text{J3-5a})$$

$$k_{sc} = 1 - \frac{1.5 T_u}{D_u T_b n_b} \quad (\text{ASD}) \quad (\text{J3-5b})$$

where

$T_a$  = required tension force using *ASD load combinations*, kips (kN)

$T_u$  = required tension force using *LRFD load combinations*, kips (kN)

$n_b$  = number of bolts carrying the applied tension

## 10. Bearing Strength at Bolt Holes

The *available bearing strength*,  $\phi R_n$  and  $R_n/\Omega$ , at bolt holes shall be determined for the *limit state of bearing* as follows:

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

The nominal bearing strength of the connected material,  $R_n$ , is determined as follows:

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

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

$$R_n = 1.2l_c t F_u \leq 2.4dt F_u \quad (\text{J3-6a})$$

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

$$R_n = 1.5l_c t F_u \leq 3.0dt F_u \quad (\text{J3-6b})$$

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

$$R_n = 1.0l_c t F_u \leq 2.0dt F_u \quad (\text{J3-6c})$$

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

where

$F_u$  = *specified minimum tensile strength* of the connected material, ksi (MPa)

$d$  = nominal bolt diameter, in. (mm)

$l_c$  = clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material, in. (mm)

$t$  = thickness of connected material, in. (mm)

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 designed to prevent slip as a serviceability limit state. The use of oversized holes and short- and long-slotted holes

parallel to the line of force is restricted to slip-critical connections per Section J3.2.

## 11. Special Fasteners

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

## 12. Tension Fasteners

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

## J4. AFFECTED ELEMENTS OF MEMBERS AND CONNECTING ELEMENTS

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

### 1. Strength of Elements in Tension

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

(a) For tensile yielding of connecting elements

$$R_n = F_y A_g \quad (\text{J4-1})$$

$$\phi = 0.90 \text{ (LRFD)}$$

$$\Omega = 1.67 \text{ (ASD)}$$

(b) For tensile rupture of connecting elements

$$R_n = F_u A_e \quad (\text{J4-2})$$

$$\phi = 0.75 \text{ (LRFD)}$$

$$\Omega = 2.00 \text{ (ASD)}$$

where

$A_e$  = *effective net area* as defined in Section D3, in.<sup>2</sup> (mm<sup>2</sup>); for bolted *splice* plates,  $A_e = A_n \leq 0.85A_g$ .

**User Note:** The effective area of the connection plate may be limited due to uneven stress distribution as calculated by methods such as the Whitmore section.

### 2. Strength of Elements in Shear

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

shear yielding and shear rupture:

(a) For shear yielding of the element:

$$R_n = 0.60F_y A_{gv} \quad (J4-3)$$

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

where

$A_{gv}$  = gross area subject to shear, in.<sup>2</sup> (mm<sup>2</sup>)

(b) For shear rupture of the element:

$$R_n = 0.6F_u A_{nv} \quad (J4-4)$$

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

where

$A_{nv}$  = net area subject to shear, in.<sup>2</sup> (mm<sup>2</sup>)

### 3. Block Shear Strength

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

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

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

where

$A_{gv}$  = gross area subject to shear, in.<sup>2</sup> (mm<sup>2</sup>)

$A_{nt}$  = *net area* subject to tension, in.<sup>2</sup> (mm<sup>2</sup>)

$A_{nv}$  = net area subject to shear, in.<sup>2</sup> (mm<sup>2</sup>)

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

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

### 4. Strength of Elements in Compression

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

(a) When  $KL/r \leq 25$

$$P_n = F_y A_g \quad (J4-6)$$

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

(b) When  $KL/r > 25$  the provisions of Chapter E apply.

## 5. Strength of Elements in Flexure

The available flexural strength of affected elements shall be the lower value obtained according to the *limit states of flexural yielding, local buckling, flexural lateral-torsional buckling and flexural rupture*.

**User Note:** The *Steel Construction Manual* provides example calculations of flexural strength of specific applications.

## J5. FILLERS

### 1. Welded

Whenever it is necessary to use fillers in joints required to transfer applied force, the fillers and the connecting welds shall conform to the requirements of J5.1a or J5.1b as applicable.

#### 1a. Thin Fillers

Fillers less than  $\frac{1}{4}$  in. (6 mm) thick shall not be used to transfer stress. When the thickness of the fillers is less than  $\frac{1}{4}$  in. (6 mm), or when the thickness of the filler is  $\frac{1}{4}$  in. (6 mm) or greater but not adequate to transfer the applied force between the connected parts, the filler shall be kept flush with the edge of the outside connected part, and the size of the weld shall be increased over the required size by an amount equal to the thickness of the filler.

#### 1b. Thick Fillers

When the thickness of the fillers is adequate to transfer the applied force between the connected parts, the filler shall extend beyond the edges of the outside connected base metal. The welds joining the outside connected base metal to the filler shall be sufficient to transmit the force to the filler and the area subjected to the applied force in the filler shall be adequate to avoid overstressing the filler. The welds joining the filler to the inside connected base metal shall be adequate to transmit the applied force.

### 2. Bolted

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

(a) The shear strength of the bolts shall be multiplied by the factor  $[1 - 0.4(t - 0.25)]$  [S.I.:  $[1 - 0.0154(t - 6)]]$  but not less than 0.85,

where  $t$  is the total thickness of the fillers;

(b) 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;

(c) The size of the joint shall be increased to accommodate a number of bolts that is equivalent to the total number required in (b) above; or

(d) The joint shall be designed to prevent *slip* in accordance with Section J3.8 using either Class B surfaces or turn of the nut tightening.

## J6. SPLICES

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

## J7. BEARING STRENGTH

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

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

The nominal bearing strength,  $R_n$ , shall be determined as follows:

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

$$R_n = 1.8 F_y A_{pb} \quad (J7-1)$$

where

$F_y$  = specified minimum yield stress, ksi (MPa)

$A_{pb}$  = projected bearing area, in.<sup>2</sup> (mm<sup>2</sup>)

(b) For *expansion rollers* and *rockers*

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

$$R_n = 1.2(F_y - 13)ld / 20 \quad (J7-2)$$



$$(S.I.: R_n = 1.2(F_y - 90)ld / 20) \quad (J7-2M)$$

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

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

$$(S.I.: R_n = 30.2(F_y - 90)l\sqrt{d} / 20) \quad (J7-3M)$$

where

$d$  = diameter, in. (mm)

$l$  = length of bearing, in. (mm)

## J8. COLUMN BASES AND BEARING ON CONCRETE

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

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

$$\phi_c = 0.65 \text{ (LRFD)} \quad \Omega_c = 2.31 \text{ (ASD)}$$

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

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

$$P_p = 0.85f'_c A_1 \quad (J8-1)$$

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

$$P_p = 0.85f'_c A_1 \sqrt{A_2 / A_1} \leq 1.7f'_c A_1 \quad (J8-2)$$

where

$A_1$  = area of steel concentrically bearing on a concrete support, in.<sup>2</sup> (mm<sup>2</sup>)

$A_2$  = maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, in.<sup>2</sup> (mm<sup>2</sup>)

## J9. ANCHOR RODS AND EMBEDMENTS

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

When columns are required to resist a horizontal force at the base plate, bearing against the concrete elements should be considered. When anchor rods are used to resist horizontal forces; hole size, anchor rod setting tolerance, and the horizontal movement of the column shall be considered in the design. Design of column bases and anchor rods for the transfer of forces to the concrete foundation shall satisfy the requirements of ACI 318 or ACI 349.

Larger oversized holes and slotted holes are permitted in base plates when adequate bearing is provided for the nut by using ASTM F844 washers or plate washers and ASTM F436 washers to bridge the hole.

**User Note:** The permitted hole sizes, corresponding washer dimensions and nuts are given in the AISC *Steel Construction Manual* and ASTM F1554.

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

## J10. FLANGES AND WEBS WITH CONCENTRATED FORCES

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

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

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

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

### 1. Flange Local Bending

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

The *design strength*,  $\phi R_n$ , and the *allowable strength*,  $R_n/\Omega$ , for the *limit state* of flange *local bending* shall be determined as follows:

$$R_n = 6.25 t_f^2 F_{yf} \quad (\text{J10-1})$$

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

where

$F_{yf}$  = specified minimum yield stress of the flange, ksi (MPa)

$t_f$  = thickness of the loaded flange, in. (mm)

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

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

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

## 2. Web Local Yielding

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

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

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

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

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

$$R_n = (5k + l_b) F_{yw} t_w \quad (\text{J10-2})$$

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

$$R_n = (2.5k + l_b) F_{yw} t_w \quad (\text{J10-3})$$

where

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

$k$  = distance from outer face of the flange to the web toe of the fillet, in. (mm)

$l_b$  = length of bearing (not less than  $k$  for end beam reactions), in. (mm)

$t_w$  = web thickness, in. (mm)

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

### 3. Web Crippling

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

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

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

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

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

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

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

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

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

- (ii) For  $N/d > 0.2$

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

where

$d$  = full nominal depth of the section, in. (mm)

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

### 4. Web Sidesway Buckling

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

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

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

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

(a) If the compression flange is restrained against rotation

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

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

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

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

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

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

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

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

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

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

- $C_r$  = 960,000 ksi (6.62x10<sup>6</sup> MPa) when  $M_u < M_y$  (LRFD)  
or  $1.5M_a < M_y$  (ASD) at the location of the force
- = 480,000 ksi (3.31x10<sup>6</sup> MPa) when  $M_u \geq M_y$  (LRFD)  
or  $1.5M_a \geq M_y$  (ASD) at the location of the force
- $M_a$  = required flexural strength using ASD load combinations, kip-in. (N-mm)
- $M_u$  = design flexural strength, using LRFD load combinations, kip-in. (N-mm)
- $b_f$  = flange width, in. (mm)
- $h$  = clear distance between flanges less the fillet or corner radius for rolled shapes; distance between adjacent lines of *fasteners* or the clear distance between flanges when welds are used for built-up shapes, in. (mm)
- $l$  = largest laterally *unbraced length* along either flange at

the point of *load*, in. (mm)

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

## 5. Web Compression Buckling

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

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

$$R_n = \frac{24t_w^3 \sqrt{EF_{yw}}}{h} \quad (\text{J10-8})$$

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

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%.

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

## 6. Web Panel Zone Shear

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

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

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

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

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

- (i) For  $P_r \leq 0.4P_c$

$$R_n = 0.60F_y d_c t_w \quad (\text{J10-9})$$

- (ii) For  $P_r > 0.4P_c$

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

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

(i) For  $P_r \leq 0.75P_c$

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

(ii) For  $P_r > 0.75P_c$

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

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

$A$  = column cross-sectional area, in.<sup>2</sup> (mm<sup>2</sup>)

$b_{cf}$  = width of column flange, in. (mm)

$d_b$  = depth of beam, in. (mm)

$d_c$  = depth of column, in. (mm)

$F_y$  = specified minimum yield stress of the column web, ksi (MPa)

$P_c = P_y$ , kips (N) (LRFD)

$P_c = 0.6P_y$ , kips (N) (ASD)

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

$P_y = F_y A$ , axial yield strength of the column, kips (N)

$t_{cf}$  = thickness of the column flange, in. (mm)

$t_w$  = thickness of column web, in. (mm)

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

See Section J10.9 for doubler plate design requirements.

## 7. Unframed Ends of Beams and Girders

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

## 8. Additional Stiffener Requirements for Concentrated Forces

Stiffeners required to resist tensile concentrated forces shall be designed in accordance with the requirements of Section J4.1 and welded to the loaded flange and the web. The welds to the flange shall be sized for the difference between the required strength and available

1475 *limit state* strength. The stiffener to web welds shall be sized to  
 1476 transfer to the web the algebraic difference in tensile force at the ends  
 1477 of the stiffener.

1478  
 1479 Stiffeners required to resist compressive concentrated forces shall be  
 1480 designed in accordance with the requirements in Section J4.4 and shall  
 1481 either bear on or be welded to the loaded flange and welded to the  
 1482 web. The welds to the flange shall be sized for the difference between  
 1483 the required strength and the applicable limit state strength. The weld  
 1484 to the web shall be sized to transfer to the web the algebraic difference  
 1485 in compression force at the ends of the stiffener. For fitted bearing  
 1486 stiffeners, see Section J7.

1487  
 1488 Transverse full depth bearing stiffeners for compressive forces applied  
 1489 to a *beam* or *plate girder* flange(s) shall be designed as axially  
 1490 compressed members (*columns*) in accordance with the requirements  
 1491 of Sections E6.2 and J4.4. The member properties shall be determined  
 1492 using an *effective length* of  $0.75h$  and a cross section composed of two  
 1493 stiffeners and a strip of the web having a width of  $25t_w$  at interior  
 1494 stiffeners and  $12t_w$  at the ends of members. The weld connecting full  
 1495 depth bearing stiffeners to the web shall be sized to transmit the  
 1496 difference in compressive force at each of the stiffeners to the web.

1497  
 1498 Transverse and *diagonal stiffeners* shall comply with the following  
 1499 additional requirements:

- 1500  
 1501 (1) The width of each stiffener plus one-half the thickness of the  
 1502 column web shall not be less than one-third of the flange or  
 1503 moment connection plate width delivering the concentrated  
 1504 force.  
 1505 (2) The thickness of a stiffener shall not be less than one-half the  
 1506 thickness of the flange or moment connection plate delivering  
 1507 the concentrated *load*, nor less than the width divided by 16.  
 1508 (3) *Transverse stiffeners* shall extend a minimum of one-half the  
 1509 depth of the member except as required in J10.5 and J10.7.

## 1510 9. Additional Doubler Plate Requirements for Concentrated Forces

1511  
 1512 *Doubler* plates required for compression strength shall be designed in  
 1513 accordance with the requirements of Chapter E.

1514  
 1515  
 1516 Doubler plates required for tensile strength shall be designed in  
 1517 accordance with the requirements of Chapter D.

1518  
 1519 Doubler plates required for shear strength (see Section J10.6) shall be  
 1520 designed in accordance with the provisions of Chapter G.

1521  
 1522 Doubler plates shall comply with the following additional require-  
 1523 ments:

- 1524 (1) The thickness and extent of the doubler plate shall provide the



- 1525 additional material necessary to equal or exceed the strength  
1526 requirements.  
1527 (2) The doubler plate shall be welded to develop the proportion of  
1528 the total force transmitted to the doubler plate.

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## CHAPTER K

### DESIGN OF HSS AND BOX MEMBER CONNECTIONS

This chapter addresses connections to *HSS* members and box sections of uniform wall thickness.

**User Note:** Connections often govern the size of HSS members, especially the thickness of truss chords, and must be considered in the initial design

The chapter is organized as follows:

- K1. Concentrated Forces on HSS
- K2. HSS-to-HSS Truss Connections
- K3. HSS-to-HSS Moment Connections
- K4. Welds to Branches

**User Note:** See also Chapter J for additional requirements for bolting to HSS. See Section J3.10(c) for through-bolts.

**User Note:** Connection dimensions must be within the limits of applicability. Limit states need only be checked when connection geometry or loading is within the parameters given in the description of the limit state.

#### K1. CONCENTRATED FORCES ON HSS

##### 1. Definitions of Parameters

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

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

$D$  = chord diameter, in. (mm)

$F_c$  = available stress in chord, ksi (MPa)  
=  $F_y$  for LRFD;  $0.6F_y$  for ASD

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

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

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

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

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

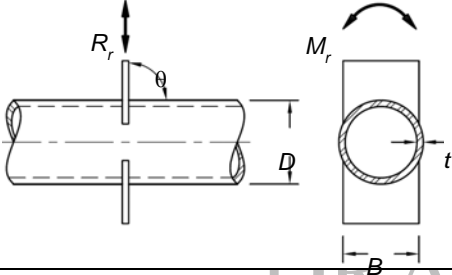
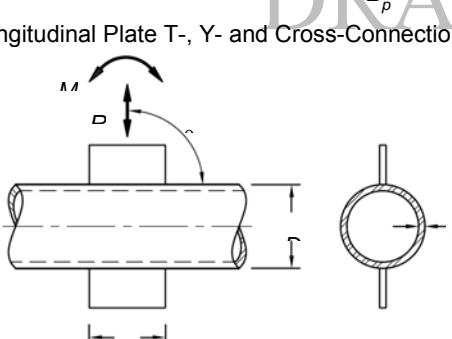
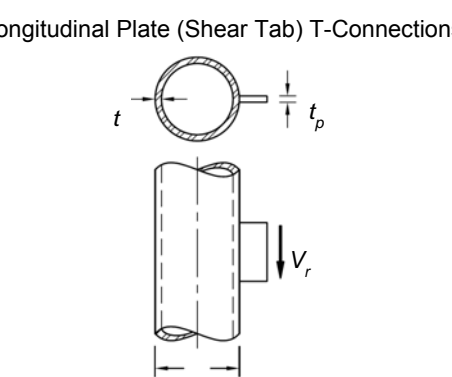
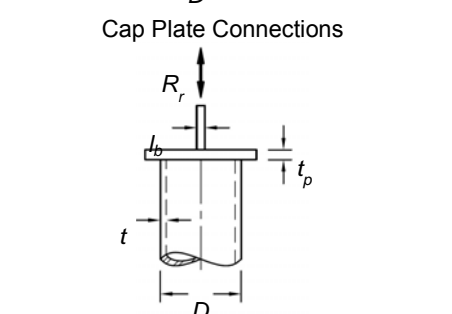
$t$  = *design wall thickness* of HSS member, in. (mm)

$t_p$  = thickness of plate, in. (mm)

##### 2. Round HSS

The available strength of connections with concentrated loads and within the limits in Table K1.1A shall be taken as shown in Table K1.1.

**Table K1.1. Available Strengths of Plate-to-Round HSS Connections**

Connection Type	Connection Available Strength	Plate Bending	
Connection Type	Connection Available Strength	In-Plane	Out-of-Plane
<p>Transverse Plate T- and Cross-Connections</p> 	<p>Limit state: HSS local yielding</p> <p>Plate Axial Load:</p> $R_n \sin \theta = F_y t^2 \left[ \frac{5.5}{1 - 0.81 \frac{B_p}{D}} \right] Q_f \quad (\text{K1-1})$ <p><math>\phi = 0.90</math> (LRFD)      <math>\Omega = 1.67</math> (ASD)</p>	—	$M_n = 0.5 B_p R_n$
<p>Longitudinal Plate T-, Y- and Cross-Connections</p> 	<p>Limit state: HSS plastification</p> <p>Plate Axial Load:</p> $R_n \sin \theta = 5.5 F_y t^2 \left[ 1 + 0.25 \frac{l_b}{D} \right] Q_f \quad (\text{K1-2})$ <p><math>\phi = 0.90</math> (LRFD)      <math>\Omega = 1.67</math> (ASD)</p>	$M_n = 0.8 l_b R_n$	—
<p>Longitudinal Plate (Shear Tab) T-Connections</p> 	<p>Limit states: plate yielding and HSS punching shear</p> <p>Plate Shear Load:</p> $t_p \leq \frac{F_u}{F_{yp}} t \quad (\text{K1-3})$	—	—
<p>Cap Plate Connections</p> 	<p>Limit state: local yielding of HSS</p> <p>Axial Load:</p> $R_n = 2 F_y t [5 t_p + l_b] \leq A F_y \quad (\text{K1-4})$ <p><math>\phi = 1.00</math> (LRFD)      <math>\Omega = 1.50</math> (ASD)</p>	—	—
<b>FUNCTIONS</b>			
<p><math>Q_f = 1</math> for HSS (connecting surface) in tension</p> <p><math>= 1.0 - 0.3U (1 + U)</math> for HSS (connecting surface) in compression</p> <p style="text-align: right;">(K1-5)</p>			

$$U = \left| \frac{P_{ro}}{AF_c} + \frac{M_{ro}}{SF_c} \right|, \text{ where } P_{ro} \text{ and } M_{ro} \text{ are determined on the side of the joint that has the lower compression stress. } P_{ro} \text{ and } M_{ro} \text{ refer to loads in the HSS.}$$

(K1-6)

$P_{ro} = P_u$  for LRFD;  $P_a$  for ASD.  $M_{ro} = M_u$  for LRFD;  $M_a$  for ASD.

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**Table K1.1A Limits of Applicability of Table K1.1**

Plate load angle:	$\theta \geq 30^\circ$
HSS wall slenderness:	$D/t \leq 50$ for T-connections under branch plate axial load or bending $D/t \leq 40$ for Cross-connections under branch plate axial load or bending $D/t \leq 0.11E/F_y$ under branch plate shear loading $D/t \leq 0.11E/F_y$ for cap plate connections in compression
Width ratio:	$0.2 < B_p/D \leq 1.0$ for transverse branch plate connections
Material strength:	$F_y \leq 52 \text{ ksi (360 MPa)}$
Ductility:	$F_y/F_u \leq 0.8$

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**3. Rectangular HSS**

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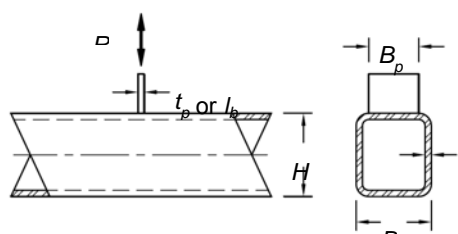
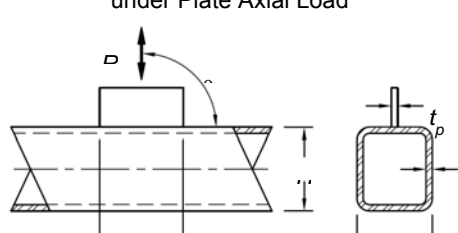
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The available strength of connections with concentrated loads and within the limits in Table K1.2A shall be taken as the lowest value of the applicable limit states shown in Table K1.2.

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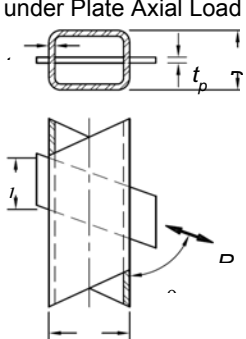
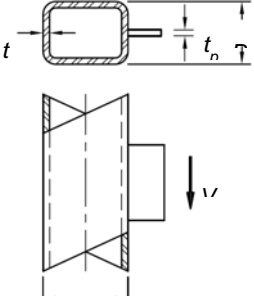
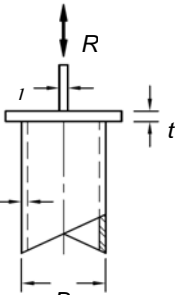
Table K1.2. Available Strengths of Plate-to-Rectangular HSS Connections

Connection Type	Connection Available Strength
<p>Transverse Plate T- and Cross-Connections, under Plate Axial Load</p>  <p>where <math>\beta = \frac{B_p}{B}</math></p>	<p>Limit state: local yielding of plate, for all <math>\beta</math></p> $R_n = \frac{10}{B/t} F_y t B_p \leq F_{yp} t_p B_p \quad (K1-7)$ <p><math>\phi = 0.95</math> (LRFD)      <math>\Omega = 1.58</math> (ASD)</p>
	<p>Limit state: HSS shear yielding (punching), when <math>0.85B \leq B_p \leq B - 2t</math></p> $R_n = 0.6 F_y t [2t_p + 2B_{ep}] \quad (K1-8)$ <p><math>\phi = 0.95</math> (LRFD)      <math>\Omega = 1.58</math> (ASD)</p>
	<p>Limit state: local yielding of HSS side walls, when <math>\beta = 1.0</math></p> $R_n = 2 F_y t [5k + l_b] \quad (K1-9)$ <p><math>\phi = 1.00</math> (LRFD)      <math>\Omega = 1.50</math> (ASD)</p>
	<p>Limit state: local crippling of HSS side walls, when <math>\beta = 1.0</math> and plate is in compression, for T-connections</p> $R_n = 1.6 t^2 \left[ 1 + \frac{3l_b}{H - 3t} \right] \sqrt{E F_y} Q_f \quad (K1-10)$ <p><math>\phi = 0.75</math> (LRFD)      <math>\Omega = 2.00</math> (ASD)</p>
	<p>Limit state: local crippling of HSS side walls, when <math>\beta = 1.0</math> and plates are in compression, for Cross-connections</p> $R_n = \left[ \frac{48 t^3}{H - 3t} \right] \sqrt{E F_y} Q_f \quad (K1-11)$ <p><math>\phi = 0.90</math> (LRFD)      <math>\Omega = 1.67</math> (ASD)</p>
<p>Longitudinal Plate T-, Y- and Cross-Connections, under Plate Axial Load</p> 	<p>Limit state: HSS wall plastification</p> $R_n \sin \theta = \frac{F_y t^2}{1 - \frac{t_p}{B}} \left[ \frac{2l_b}{B} + 4 \sqrt{1 - \frac{t_p}{B}} Q_f \right] \quad (K1-12)$ <p><math>\phi = 1.00</math> (LRFD)      <math>\Omega = 1.50</math> (ASD)</p>

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Table K1.2 (continued)

Connection Type	Connection Available Strength
<p>Longitudinal Through Plate T- and Y-Connections, under Plate Axial Load</p> 	<p>Limit state: HSS wall plastification</p> $R_n \sin \theta = \frac{2F_y t^2}{1 - \frac{t_p}{B}} \left[ \frac{2l_b}{B} + 4\sqrt{1 - \frac{t_p}{B}} Q_f \right] \quad (\text{K1-13})$ <p><math>\phi = 1.00</math> (LRFD)      <math>\Omega = 1.50</math> (ASD)</p>
<p>Longitudinal Plate (Shear Tab) T-Connections, under Plate Shear Load</p> 	<p>Limit states: plate yielding and HSS punching shear</p> $t_p \leq \frac{F_u}{F_{yp}} t \quad (\text{K1-3})$
<p>Cap Plate Connections, under Axial Load</p> 	<p>Limit state: local yielding of side walls</p> $R_n = 2F_y t [5t_p + l_b], \text{ when } (5t_p + l_b) < B$ $R_n = AF_y, \text{ when } (5t_p + l_b) \geq B \quad (\text{K1-14})$ <p><math>\phi = 1.00</math> (LRFD)      <math>\Omega = 1.50</math> (ASD)</p>
	<p>Limit state: local crippling of side walls, when plate is in compression</p> $R_n = 1.6t^2 \left[ 1 + \frac{6l_b}{B} \left( \frac{t}{t_p} \right)^{1.5} \right] \sqrt{EF_y \frac{t_p}{t}}, \text{ when } (5t_p + l_b) < B \quad (\text{K1-15})$ <p><math>\phi = 0.75</math> (LRFD)      <math>\Omega = 2.00</math> (ASD)</p>

FUNCTIONS	
$Q_f = 1$ for HSS (connecting surface) in tension	
$= 1.3 - 0.4 \frac{U}{\beta} \leq 1.0$ for HSS (connecting surface) in compression, for transverse plate connections	(K1-16)
$= \sqrt{1 - U^2}$ for HSS (connecting surface) in compression, for longitudinal plate and longitudinal through plate connections	(K1-17)
$U = \left  \frac{P_{ro}}{AF_c} + \frac{M_{ro}}{SF_c} \right $ , where $P_{ro}$ and $M_{ro}$ are determined on the side of the joint that has the higher compression stress. $P_{ro}$ and $M_{ro}$ refer to loads in the HSS. $P_{ro} = P_u$ for LRFD; $P_a$ for ASD. $M_{ro} = M_u$ for LRFD; $M_a$ for ASD.	(K1-6)
$B_{ep} = \frac{10B_p}{B/t} \leq B_p$	(K1-18)
$k = \text{outside corner radius of HSS} \geq 1.5t$	

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**Table K1.2A Limits of Applicability of Table K1.2**

Plate load angle:	$\theta \geq 30^\circ$
HSS wall slenderness:	$B/t$ or $H/t \leq 35$ for loaded wall, for transverse branch plate connections $B/t$ or $H/t \leq 40$ for loaded wall, for longitudinal branch plate and through plate connections
	$(B - 3t)/t$ or $(H - 3t)/t \leq 1.40\sqrt{E/F_y}$ for loaded wall, for branch plate shear loading
Width ratio:	$0.25 \leq B_p/B \leq 1.0$ for transverse branch plate connections
Material strength:	$F_y \leq 52 \text{ ksi} \quad (360 \text{ MPa})$
Ductility:	$F_y/F_u \leq 0.8$

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## K2. HSS-TO-HSS TRUSS CONNECTIONS

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

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

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

- (c) When the punching load ( $P_r \sin\theta$ ) is transmitted through the chord member and is equilibrated by branch member(s) on the opposite side, the connection shall be classified as a *cross-connection*.
- (d) When a connection has more than two primary branch members, or branch members in more than one plane, the connection shall be classified as a general or multiplanar connection.

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

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

### 1. Definitions of Parameters

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

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

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

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

$F_c$  = available stress in chord, ksi (MPa)

=  $F_y$  for LRFD;  $0.6F_y$  for ASD

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

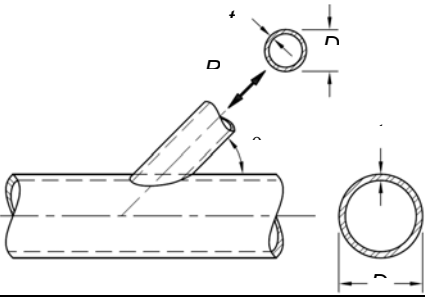
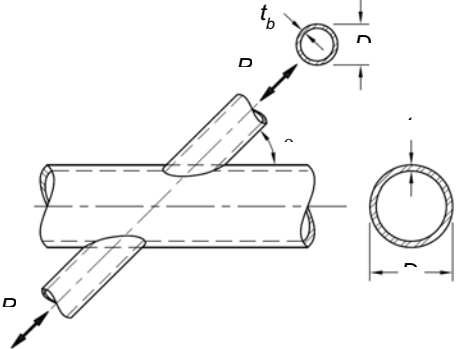
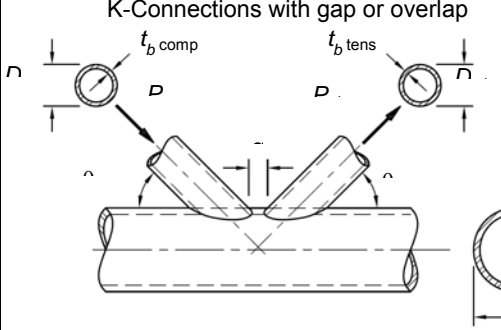
$F_{yb}$  = *specified minimum yield stress* of HSS branch member material, ksi (MPa)

$F_u$  = specified minimum tensile strength of HSS material, ksi (MPa)  
 $H$  = overall height of rectangular HSS main member, measured in the plane of the connection, in. (mm)  
 $H_b$  = overall height of rectangular HSS branch member, measured in the plane of the connection, in. (mm)  
 $O_v = q/p \times 100, \%$   
 $e$  = eccentricity in a truss connection, positive being away from the branches, in. (mm)  
 $g$  = gap between toes of branch members in a gapped K- connection, neglecting the welds, in. (mm)  
 $l_p$  = projected length of the overlapping branch on the chord, in. (mm)  
 $l_{ov}$  = overlap length measured along the connecting face of the chord beneath the two branches, in. (mm)  
 $t$  = design wall thickness of HSS main member, in. (mm)  
 $t_b$  = design wall thickness of HSS branch member, in. (mm)  
 $\beta$  = 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  
 $\beta_{eff}$  = the effective width ratio; the sum of the perimeters of the two branch members in a K-connection divided by eight times the chord width  
 $\gamma$  = 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  
 $\eta$  = 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  $N=H_b/\sin\theta$   
 $\theta$  = acute angle between the branch and chord (degrees)  
 $\zeta$  = the gap ratio; the ratio of the gap between the branches of a gapped K-connection to the width of the chord =  $g/B$  for rectangular HSS

## 2. Round HSS

The available strength of HSS to HSS truss connections within the limits in Table K2.1A shall be taken as the lowest value of the applicable limit states shown in Table K2.1.

**Table K2.1. Available Strengths of Round HSS-to-HSS Truss Connections**

Connection Type	Connection Available Axial Strength
<p>General Check</p> <p>For T-, Y-, Cross- and K-Connections with gap, when <math>D_{b(tens/comp)} &lt; (D - 2t)</math></p>	<p>Limit state: shear yielding (punching)</p> $P_n = 0.6F_y t \pi D_b \left[ \frac{1 + \sin \theta}{2 \sin^2 \theta} \right] \quad (K2-1)$ <p><math>\phi = 0.95</math> (LRFD)    <math>\Omega = 1.58</math> (ASD)</p>
<p>T- and Y-Connections</p> 	<p>Limit state: chord plastification</p> $P_n \sin \theta = F_y t^2 \left[ 3.1 + 15.6 \beta^2 \right] \gamma^{0.2} Q_f \quad (K2-2)$ <p><math>\phi = 0.90</math> (LRFD)    <math>\Omega = 1.67</math> (ASD)</p>
<p>Cross-Connections</p> 	<p>Limit state: chord plastification</p> $P_n \sin \theta = F_y t^2 \left[ \frac{5.7}{1 - 0.8 \beta} \right] Q_f \quad (K2-3)$ <p><math>\phi = 0.90</math> (LRFD)    <math>\Omega = 1.67</math> (ASD)</p>
<p>K-Connections with gap or overlap</p> 	<p>Limit state: chord plastification</p> $(P_n \sin \theta)_{\text{compression branch}} = F_y t^2 \left[ 2.0 + 11.33 \frac{D_{b \text{ comp}}}{D} \right] Q_g Q_f \quad (K2-4)$ $(P_n \sin \theta)_{\text{tension branch}} = (P_n \sin \theta)_{\text{compression branch}} \quad (K2-5)$ <p><math>\phi = 0.90</math> (LRFD)    <math>\Omega = 1.67</math> (ASD)</p>

FUNCTIONS	
$Q_f = 1$ for chord (connecting surface) in tension	
$= 1.0 - 0.3U (1 + U)$ for HSS (connecting surface) in compression	(K1-5)
$U = \left  \frac{P_{ro}}{AF_c} + \frac{M_{ro}}{SF_c} \right $ , where $P_{ro}$ and $M_{ro}$ are determined on the side of the joint that has the lower compression stress. $P_{ro}$ and $M_{ro}$ refer to loads in the chord. $P_{ro} = P_u$ for LRFD; $P_a$ for ASD. $M_{ro} = M_u$ for LRFD; $M_a$ for ASD.	(K1-6)
$Q_g = \gamma^{0.2} \left[ 1 + \frac{0.024\gamma^{1.2}}{\exp\left(\frac{0.5g}{t} - 1.33\right) + 1} \right]$	(K2-6)

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**Table K2.1A Limits of Applicability of Table K2.1**

Joint eccentricity:	$-0.55 \leq e/D \leq 0.25$ for K-connections
Branch angle:	$\theta \geq 30^\circ$
Chord wall slenderness:	$D/t \leq 50$ for T-, Y- and K-connections $D/t \leq 40$ for Cross-connections
Branch wall slenderness:	$D_b/t_b \leq 50$ for tension branch $D_b/t_b \leq 0.05E/F_{yb}$ for compression branch
Width ratio:	$0.2 < D_b/D \leq 1.0$ for T-, Y-, Cross- and overlapped K-connections $0.4 \leq D_b/D \leq 1.0$ for gapped K-connections
Gap:	$g \geq t_{b \text{ comp}} + t_{b \text{ tens}}$ for gapped K-connections
Overlap:	$25\% \leq O_v \leq 100\%$ for overlapped K-connections
Branch thickness:	$t_{b \text{ overlapping}} \leq t_{b \text{ overlapped}}$ for branches in overlapped K-connections
Material strength:	$F_y$ and $F_{yb} \leq 52 \text{ ksi}$ (360 MPa)
Ductility:	$F_y/F_u$ and $F_{yb}/F_{ub} \leq 0.8$

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**3. Rectangular HSS**

The available strength of HSS-to-HSS truss connections within the limits in Table K2.2A shall be taken as the lowest value of the applicable limit states shown in Table K2.2.

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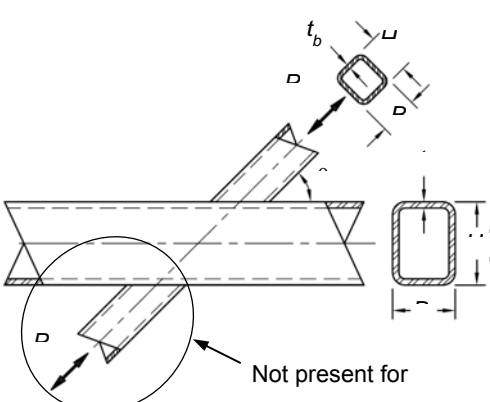
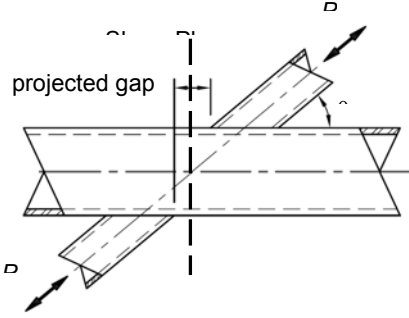
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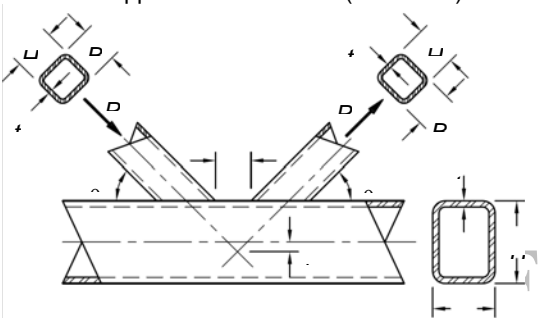
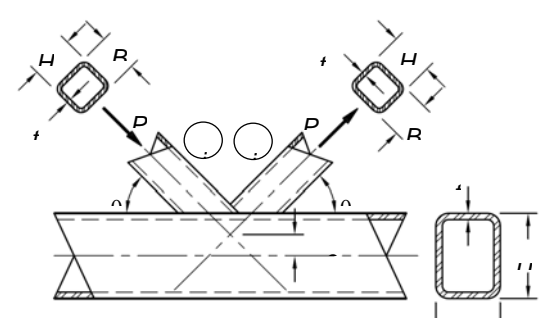
**Table K2.2. Available Strengths of Rectangular HSS-to-HSS Truss Connections**

Connection Type	Connection Available Axial Strength
<p>T-, Y- and Cross-Connections</p>  <p>Not present for</p> <p>Case for checking limit state of shear of chord side walls</p> 	<p>Limit state: chord wall plastification, when <math>\beta \leq 0.85</math></p> $P_n \sin \theta = F_y t^2 \left[ \frac{2\eta}{(1-\beta)} + \frac{4}{\sqrt{1-\beta}} \right] Q_f \quad (\text{K2-7})$ <p><math>\phi = 1.00</math> (LRFD)      <math>\Omega = 1.50</math> (ASD)</p>
	<p>Limit state: shear yielding (punching), when <math>0.85 &lt; \beta \leq 1 - 1/\gamma</math> or <math>B/t &lt; 10</math></p> $P_n \sin \theta = 0.6 F_y t B [2\eta + 2\beta_{eop}] \quad (\text{K2-8})$ <p><math>\phi = 0.95</math> (LRFD)      <math>\Omega = 1.58</math> (ASD)</p>
	<p>Limit state: local yielding of chord side walls, when <math>\beta = 1.0</math></p> $P_n \sin \theta = 2 F_y t [5k + l_b] \quad (\text{K2-9})$ <p><math>\phi = 1.00</math> (LRFD)      <math>\Omega = 1.50</math> (ASD)</p>
	<p>Limit state: local crippling of chord side walls, when <math>\beta = 1.0</math> and branch is in compression, for T- or Y-connections</p> $P_n \sin \theta = 1.6 t^2 \left[ 1 + \frac{3l_b}{H - 3t} \right] \sqrt{E F_y} Q_f \quad (\text{K2-10})$ <p><math>\phi = 0.75</math> (LRFD)      <math>\Omega = 2.00</math> (ASD)</p>
	<p>Limit state: local crippling of chord side walls, when <math>\beta = 1.0</math> and branches are in compression, for Cross-connections</p> $P_n \sin \theta = \left[ \frac{48 t^3}{H - 3t} \right] \sqrt{E F_y} Q_f \quad (\text{K2-11})$ <p><math>\phi = 0.90</math> (LRFD)      <math>\Omega = 1.67</math> (ASD)</p>
	<p>Limit state: local yielding of branch/branches due to uneven load distribution, when <math>\beta &gt; 0.85</math></p> $P_n = F_y t_b [2H_b + 2b_{eoi} - 4t_b] \quad (\text{K2-12})$ <p><math>\phi = 0.95</math> (LRFD)      <math>\Omega = 1.58</math> (ASD)</p> $b_{eoi} = \frac{10}{B/t} \left( \frac{F_y t}{F_y t_b} \right) B_b \leq B_b \quad (\text{K2-13})$
Gapped K-Connections	<p>Limit State: shear of chord side walls, for Cross-connections with <math>\theta &lt; 90^\circ</math> and where a projected gap is created (see figure). Determine <math>P_n \sin \theta</math> in accordance with Spec. Section G5.</p>

	$P_n \sin \theta = F_y t^2 \left[ 9.8 \beta_{eff} \gamma^{0.5} \right] Q_f \tag{K2-14}$
	$\phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)}$
	<div>Limit state: shear yielding (punching), when <math>B_b &lt; B - 2t</math></div> <div>Do not check for square branches</div> $P_n \sin \theta = 0.6 F_y t B \left[ 2\eta + \beta + \beta_{eop} \right] \tag{K2-15}$ $\phi = 0.95 \text{ (LRFD)} \quad \Omega = 1.58 \text{ (ASD)}$

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Table K2.2 (continued)

Connection Type	Connection Available Axial Strength
<p>Gapped K-Connections (continued)</p> 	<p>Limit state: shear of chord side walls in the gap region. Determine <math>P_n \sin \theta</math> in accordance with Spec. Section G5. Do not check for square chords.</p>
	<p>Limit state: local yielding of branch/branches due to uneven load distribution. Do not check for square branches or if <math>B/t \geq 15</math></p> $P_n = F_{yb} t_b [2H_b + B_b + b_{eoi} - 4t_b] \quad (K2-16)$ $\phi = 0.95 \text{ (LRFD)} \quad \Omega = 1.58 \text{ (ASD)}$ $b_{eoi} = \frac{10}{B/t} \left( \frac{F_y t}{F_{yb} t_b} \right) B_b \leq B_b \quad (K2-13)$
<p>Overlapped K-Connections</p>  <p>Note that the force arrows shown for overlapped K-connections may be reversed; <i>i</i> and <i>j</i> control member identification</p>	<p>Limit state: local yielding of branch/branches due to uneven load distribution.</p> $\phi = 0.95 \text{ (LRFD)} \quad \Omega = 1.58 \text{ (ASD)}$ <p>when <math>25\% \leq O_v &lt; 50\%</math> :</p> $P_{n, \text{overlapping branch}} = F_{yb} t_{bi} \left[ \frac{O_v}{50} (2H_{bi} - 4t_{bi}) + b_{eoi} + b_{eov} \right] \quad (K2-17)$ <p>when <math>50\% \leq O_v &lt; 80\%</math> :</p> $P_{n, \text{overlapping branch}} = F_{yb} t_{bi} [2H_{bi} - 4t_{bi} + b_{eoi} + b_{eov}] \quad (K2-18)$ <p>when <math>80\% \leq O_v \leq 100\%</math> :</p> $P_{n, \text{overlapping branch}} = F_{yb} t_{bi} [2H_{bi} - 4t_{bi} + B_{bi} + b_{eov}] \quad (K2-19)$ $b_{eoi} = \frac{10}{B/t} \left( \frac{F_y t}{F_{yb} t_{bi}} \right) B_{bi} \leq B_{bi} \quad (K2-20)$ $b_{eov} = \frac{10}{B_{bj}/t_{bj}} \left( \frac{F_{yb} t_{bj}}{F_{yb} t_{bi}} \right) B_{bi} \leq B_{bi} \quad (K2-21)$ <p>Subscript <i>i</i> refers to the overlapping branch Subscript <i>j</i> refers to the overlapped branch</p> $P_{n, \text{overlapped branch}} = P_{n, \text{overlapping branch}} \left( \frac{A_{bj} F_{ybj}}{A_{bi} F_{ybi}} \right) \quad (K2-22)$

FUNCTIONS	
$Q_f = 1$ for chord (connecting surface) in tension	
$= 1.3 - 0.4 \frac{U}{\beta} \leq 1.0$ for chord (connecting surface) in compression, for T-, Y-, and Cross-connections	(K1-16)
$= 1.3 - 0.4 \frac{U}{\beta_{eff}} \leq 1.0$ for chord (connecting surface) in compression, for gapped K-connections	(K2-23)
$U = \left  \frac{P_{ro}}{AF_c} + \frac{M_{ro}}{SF_c} \right $ , where $P_{ro}$ and $M_{ro}$ are determined on the side of the joint that has the higher compression stress. $P_{ro}$ and $M_{ro}$ refer to loads in the chord. $P_{ro} = P_u$ for LRFD; $P_a$ for ASD. $M_{ro} = M_u$ for LRFD; $M_a$ for ASD.	(K1-6)
$\beta_{eff} = \left[ (B_b + H_b)_{\text{compression branch}} + (B_b + H_b)_{\text{tension branch}} \right] / 4B$	(K2-24)
$\beta_{eop} = \frac{5\beta}{\gamma} \leq \beta$	(K2-25)

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**Table K2.2A Limits of Applicability of Table K2.2**

Joint eccentricity:	$-0.55 \leq e/H \leq 0.25$ for K-connections
Branch angle:	$\theta \geq 30^\circ$
Chord wall slenderness:	$B/t$ and $H/t \leq 35$ for gapped K-connections and T-, Y- and Cross-connections
Branch wall slenderness:	$B/t \leq 30$ for overlapped K-connections
	$H/t \leq 35$ for overlapped K-connections
	$B_b/t_b$ and $H_b/t_b \leq 35$ for tension branch
	$\leq 1.25 \sqrt{\frac{E}{F_{yb}}}$ for compression branch of gapped K-, T-, Y- and Cross-connections
	$\leq 35$ for compression branch of gapped K-, T-, Y- and Cross-connections
	$\leq 1.1 \sqrt{\frac{E}{F_{yb}}}$ for compression branch of overlapped K-connections
Width ratio:	$B_b/B$ and $H_b/B \geq 0.25$ for T-, Y-, Cross- and overlapped K-connections
Aspect ratio:	$0.5 \leq H_b/B_b \leq 2.0$ and $0.5 \leq H/B \leq 2.0$
Overlap:	$25\% \leq O_v \leq 100\%$ for overlapped K-connections
Branch width ratio:	$B_{bi}/B_{bj} \geq 0.75$ for overlapped K-connections, where subscript i refers to the overlapping branch and subscript j refers to the overlapped branch
Branch thickness ratio:	$t_{bi}/t_{bj} \leq 1.0$ for overlapped K-connections, where subscript i refers to the overlapping branch and subscript j refers to the overlapped branch
Material strength:	$F_y$ and $F_{yb} \leq 52$ ksi (360 MPa)
Ductility:	$F_y/F_u$ and $F_{yb}/F_{ub} \leq 0.8$
Additional limits for gapped K-connections	
Width ratio:	$\frac{B_b}{B}$ and $\frac{H_b}{B} \geq 0.1 + \frac{\gamma}{50}$
	$\beta_{eff} \geq 0.35$
Gap ratio:	$\zeta = g/B \geq 0.5(1 - \beta_{eff})$
Gap:	$g \geq t_{b \text{ compression branch}} + t_{b \text{ tension branch}}$
Branch size:	smaller $B_b \geq 0.63(\text{larger } B_b)$ , if both branches are square
Note: Maximum gap size will be controlled by the $e/H$ limit. If gap is large, treat as two Y-connections.	

**K3. HSS-TO-HSS MOMENT CONNECTIONS**

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

A connection shall be classified as:

- (a) A *T-connection* when there is one branch and it is perpendicular to the chord and as a *Y-connection* when there is one branch but not perpendicular to the chord.
- (b) A *cross-connection* when there is a branch on each (opposite) side of

the chord.

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

## 1. Definitions of Parameters

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

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

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

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

$F_c$  = Available stress in chord, ksi (MPa)

=  $F_y$  for LRFD;  $0.6F_y$  for ASD

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

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

$F_u$  = ultimate strength of HSS member, ksi (MPa)

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

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

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

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

$\beta$  = 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

$\gamma$  = 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

$\eta$  = 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  $N=H_b/\sin\theta$

$\theta$  = acute angle between the branch and chord (degrees)

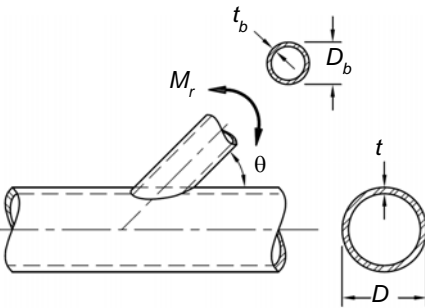
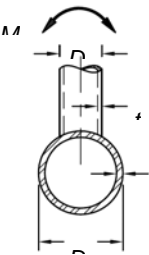
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## **2. Round HSS**

The available strength of moment connections within the limits of Table K3.1A shall be taken as the lowest value of the applicable limit states shown in Table K3.1.

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**Table K3.1. Available Capacities of Round HSS-to-HSS Moment Connections**

Connection Type	Connection Available Moment Capacity
<p>Branch(es) under In-Plane Bending T-, Y- and Cross-Connections</p> 	<p>Limit state: chord plastification</p> $M_n \sin \theta = 5.39 F_y t^{2.5} \beta D_b Q_f \quad (\text{K3-1})$ <p><math>\phi = 0.90</math> (LRFD)      <math>\Omega = 1.67</math> (ASD)</p> <hr/> <p>Limit state: shear yielding (punching), when <math>D_b &lt; (D - 2t)</math></p> $M_n = 0.6 F_y t D_b^2 \left[ \frac{1 + 3 \sin \theta}{4 \sin^2 \theta} \right] \quad (\text{K3-2})$ <p><math>\phi = 0.95</math> (LRFD)      <math>\Omega = 1.58</math> (ASD)</p>
<p>Branch(es) under Out-of-Plane Bending T-, Y- and Cross-Connections</p> 	<p>Limit state: chord plastification</p> $M_n \sin \theta = F_y t^2 D_b \left[ \frac{3.0}{1 - 0.81 \beta} \right] Q_f \quad (\text{K3-3})$ <p><math>\phi = 0.90</math> (LRFD)      <math>\Omega = 1.67</math> (ASD)</p> <hr/> <p>Limit state: shear yielding (punching), when <math>D_b &lt; (D - 2t)</math></p> $M_n = 0.6 F_y t D_b^2 \left[ \frac{3 + \sin \theta}{4 \sin^2 \theta} \right] \quad (\text{K3-4})$ <p><math>\phi = 0.95</math> (LRFD)      <math>\Omega = 1.58</math> (ASD)</p>

For T-, Y- and Cross-connections, with branch(es) under combined axial load, in-plane bending and out-of-plane bending, or any combination of these load effects:

$$\text{LRFD: } \left[ P_u / \phi P_n \right] + \left[ M_{r-ip} / \phi M_{n-ip} \right]^2 + \left[ M_{r-op} / \phi M_{n-op} \right] \leq 1.0 \quad (\text{K3-5})$$

$$\text{ASD: } \left[ P_a / (P_n / \Omega) \right] + \left[ M_{r-ip} / (M_{n-ip} / \Omega) \right]^2 + \left[ M_{r-op} / (M_{n-op} / \Omega) \right] \leq 1.0 \quad (\text{K3-6})$$

$\phi P_n$  = design strength (or  $P_n / \Omega$  = allowable strength) obtained from Table K2.1

$\phi M_{n-ip}$  = design strength (or  $M_{n-ip} / \Omega$  = allowable strength) for in-plane bending

$\phi M_{n-op}$  = design strength (or  $M_{n-op} / \Omega$  = allowable strength) for out-of-plane bending

$M_{r-ip}$  =  $M_{u-ip}$  for LRFD;  $M_{a-ip}$  for ASD

$M_{r-op}$  =  $M_{u-op}$  for LRFD;  $M_{a-op}$  for ASD

### FUNCTIONS

$Q_f = 1$  for chord (connecting surface) in tension

$= 1.0 - 0.3U(1+U)$  for chord (connecting surface) in compression (K1-5)

$U = \left| \frac{P_{ro}}{AF_c} + \frac{M_{ro}}{SF_c} \right|$ , where  $P_{ro}$  and  $M_{ro}$  refer to loads in the chord.  $P_{ro} = P_u$  for LRFD;  $P_a$  for ASD. (K1-6)  
 $M_{ro} = M_u$  for LRFD;  $M_a$  for ASD.

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**Table K3.1A Limits of Applicability of Table K3.1**

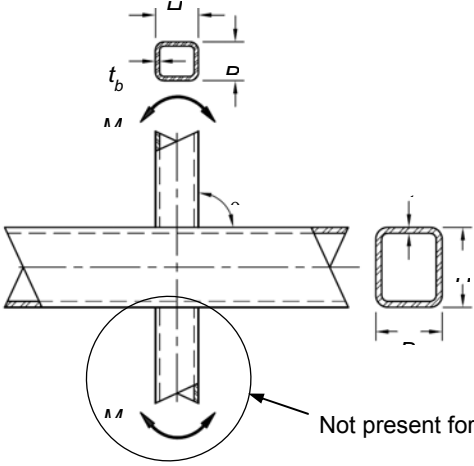
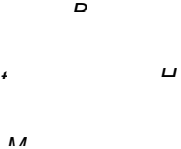
Branch angle:	$\theta \geq 30^\circ$
Chord wall slenderness:	$D/t \leq 50$ for T- and Y-connections $D/t \leq 40$ for Cross-connections
Branch wall slenderness:	$D_b/t_b \leq 50$ $D_b/t_b \leq 0.05E/F_{yb}$
Width ratio:	$0.2 < D_b/D \leq 1.0$
Material strength:	$F_y$ and $F_{yb} \leq 52 \text{ ksi}$ (360 MPa)
Ductility:	$F_y/F_u$ and $F_{yb}/F_{ub} \leq 0.8$

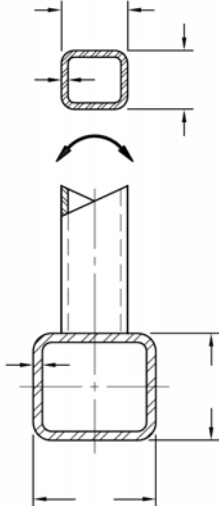
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### 3. Rectangular HSS

The available strength of moment connections within the limits of Table K3.2A shall be taken as the lowest value of the applicable limit states shown in Table K3.2.

**Table K3.2. Available Capacities of Rectangular HSS-to-HSS Moment Connections**

Connection Type	Connection Available Moment Capacity
Branch(es) under In-Plane Bending T- and Cross-Connections 	Limit state: chord wall plastification, when $\beta \leq 0.85$ $M_n = F_y t^2 H_b \left[ \frac{1}{2\eta} + \frac{2}{\sqrt{1-\beta}} + \frac{\eta}{(1-\beta)} \right] Q_f \quad (K3-7)$ $\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)
	Limit state: sidewall local yielding, when $\beta > 0.85$ $M_n = 0.5 F_y t (H_b + 5t)^2 \quad (K3-8)$ $\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)
	Limit state: local yielding of branch/branches due to uneven load distribution, when $\beta > 0.85$ $M_n = F_{yb} \left[ Z_b - \left( 1 - \frac{b_{eoi}}{B_b} \right) B_b H_b t_b \right] \quad (K3-9)$ $\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)
Branch(es) under Out-of-Plane Bending T- and Cross-Connections 	Limit state: chord wall plastification, when $\beta \leq 0.85$ $M_n = F_y t^2 \left[ \frac{0.5 H_b (1+\beta)}{(1-\beta)} + \sqrt{\frac{2 B B_b (1+\beta)}{(1-\beta)}} \right] Q_f \quad (K3-10)$ $\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)
	Limit state: sidewall local yielding, when $\beta > 0.85$ $M_n = F_y t (B - t) (H_b + 5t) \quad (K3-11)$ $\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)
	Limit state: local yielding of branch/branches due to uneven load distribution, when $\beta > 0.85$ $M_n = F_{yb} \left[ Z_b - 0.5 \left( 1 - \frac{b_{eoi}}{B_b} \right)^2 B_b^2 t_b \right] \quad (K3-12)$ $\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)

	<p>Limit state: chord distortional failure, for T-connections and unbalanced Cross-connections</p> $M_n = 2F_y t \left[ H_b t + \sqrt{B H t (B + H)} \right] \quad (\text{K3-13})$ <p><math>\phi = 1.00</math> (LRFD)      <math>\Omega = 1.50</math> (ASD)</p>
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**Table K3.2 (continued)**

For T- and Cross-connections, with branch(es) under combined axial load, in-plane bending and out-of-plane bending, or any combination of these load effects:

$$\text{LRFD: } \left[ P_u / \phi P_n \right] + \left[ M_{r-ip} / \phi M_{n-ip} \right] + \left[ M_{r-op} / \phi M_{n-op} \right] \leq 1.0 \quad (\text{K3-14})$$

$$\text{ASD: } \left[ P_a / (P_n / \Omega) \right] + \left[ M_{r-ip} / (M_{n-ip} / \Omega) \right] + \left[ M_{r-op} / (M_{n-op} / \Omega) \right] \leq 1.0 \quad (\text{K3-15})$$

$\phi P_n$  = design strength (or  $P_n / \Omega$  = allowable strength) obtained from Table K2.2,

$\phi M_{n-ip}$  = design strength (or  $M_{n-ip} / \Omega$  = allowable strength) for in-plane bending (above),

$\phi M_{n-op}$  = design strength (or  $M_{n-op} / \Omega$  = allowable strength) for out-of-plane bending (above)

$M_{r-ip} = M_{u-ip}$  for LRFD;  $M_{a-ip}$  for ASD

$M_{r-op} = M_{u-op}$  for LRFD;  $M_{a-op}$  for ASD

### FUNCTIONS

$Q_f = 1$  for chord (connecting surface) in tension

$$= 1.3 - 0.4 \frac{U}{\beta} \leq 1.0 \quad \text{for chord (connecting surface) in compression} \quad (\text{K1-16})$$

$$U = \left| \frac{P_{ro}}{AF_c} + \frac{M_{ro}}{SF_c} \right|, \quad \text{where } P_{ro} \text{ and } M_{ro} \text{ refer to loads in the chord. } P_{ro} = P_u \text{ for LRFD; } P_a \text{ for ASD.} \quad (\text{K1-6})$$

$$M_{ro} = M_u \text{ for LRFD; } M_a \text{ for ASD.}$$

$F_y^* = F_y$  for T-connections and  $= 0.8F_y$  for Cross-connections

$$b_{eoi} = \frac{10}{B/t} \left( \frac{F_y t}{F_{yb} t_b} \right) B_b \leq B_b \quad (\text{K2-13})$$

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**Table K3.2A Limits of Applicability of Table K3.2**

Branch angle:	$\theta \cong 90^\circ$
Chord wall slenderness:	$B/t$ and $H/t \leq 35$
Branch wall slenderness:	$B_b/t_b$ and $H_b/t_b \leq 35$
	$\leq 1.25 \sqrt{\frac{E}{F_{yb}}}$
Width ratio:	$B_b/B \geq 0.25$
Aspect ratio:	$0.5 \leq H_b/B_b \leq 2.0$ and $0.5 \leq H/B \leq 2.0$
Material strength:	$F_y$ and $F_{yb} \leq 52 \text{ ksi (360 MPa)}$
Ductility:	$F_y/F_u$ and $F_{yb}/F_{ub} \leq 0.8$

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## K4. WELDS TO BRANCHES

The non-uniformity 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 accomplished by determining the available strength of branch connection welds as follows:

$$R_n = F_w t_w l_e \quad (\text{K4-1})$$

$$M_{n-ip} = F_w t_w S_{ip} \quad (\text{K4-2})$$

$$M_{n-op} = F_w t_w S_{op} \quad (\text{K4-3})$$

For fillet welds

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

For partial-joint-penetration groove welds

$$\phi = 0.80 \text{ (LRFD)} \quad \Omega = 1.88 \text{ (ASD)}$$

where

$F_w$  = nominal strength of the weld metal per unit area (Chapter J) with no increase in strength due to directionality of load, ksi (MPa)

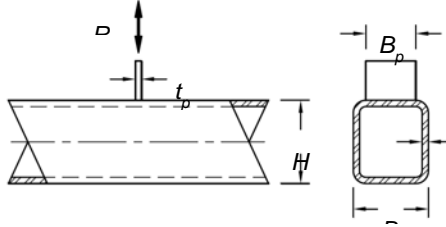
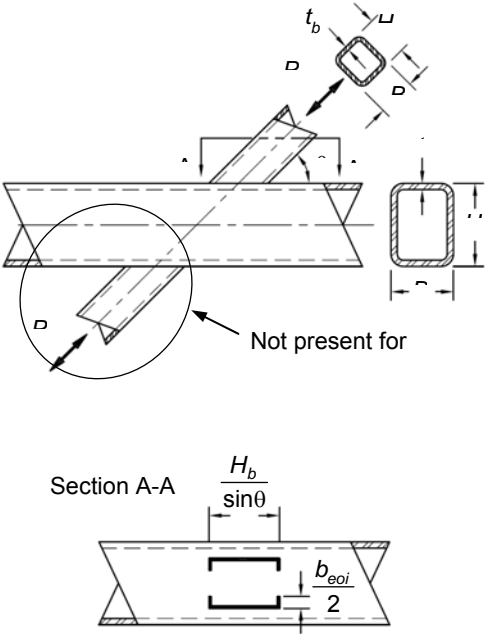
$S_{ip}$  = weld effective elastic section modulus per unit length for in-plane bending (Table K4.1), in.<sup>2</sup> (mm<sup>2</sup>)

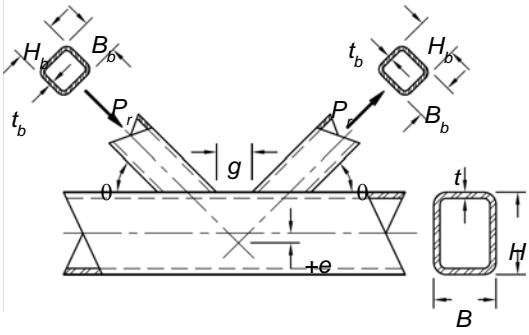
$S_{op}$  = weld effective elastic section modulus per unit length for out-of-plane bending (Table K4.1), in.<sup>2</sup> (mm<sup>2</sup>)

$l_e$  = total effective weld of groove and fillet welds to rectangular HSS. For weld strength calculations, in. (mm)

$t_w$  = smallest effective weld throat thickness around the perimeter of branch or plate, in. (mm)

**Table K4.1. Weld Capacity of Rectangular HSS-to-HSS Truss Connections**

Connection Type	Connection Weld Strength
<p>Transverse Plate T- and Cross-Connections under Plate Axial Load</p> 	<p>Effective Weld Properties</p> $l_e = 2 \left( \frac{10}{B/t} \right) \left( \frac{F_y t}{F_{yp} t_p} \right) B_p \leq 2B_p \quad (\text{K4-1})$ <p>where <math>l_e</math> = total effective weld length for welds on both sides of the transverse plate</p>
<p>T-, Y-, and Cross-Connections under Branch Axial Load or Bending</p>  <p>Not present for</p> <p>Section A-A</p>	<p>Effective Weld Properties</p> $l_e = \frac{2H_b}{\sin\theta} + 2b_{eoi} \quad (\text{K4-2})$ $S_{ip} = 0.333 \left( \frac{H_b}{\sin\theta} \right)^2 + b_{eoi} \left( \frac{H_b}{\sin\theta} \right) \quad (\text{K4-3})$ $S_{op} = \left( \frac{H_b}{\sin\theta} \right) B_b + 0.333 B_b^2 - 0.333 (B_b - b_{eoi})^2 \quad (\text{K4-4})$ $b_{eoi} = \frac{10}{B/t} \left( \frac{F_y t}{F_{yp} t_b} \right) B_b \leq B_b \quad (\text{K2-13})$ <p>when <math>\beta &gt; 0.85</math> or <math>\theta &gt; 50^\circ</math>, <math>b_{eoi}</math> shall not exceed <math>2t</math> and when <math>\beta = 1.0</math>, <math>B_b = B_b - t</math></p>
<p>Effective Weld Gapped K-Connections under Branch Axial Load</p>	<p>Effective Weld Properties</p> <p>when <math>\theta \leq 50^\circ</math>:</p> $l_e = \frac{2(H_b - 1.2t_b)}{\sin\theta} + 2(B_b - 1.2t_b) \quad (\text{K4-5})$ <p>when <math>\theta \geq 60^\circ</math>:</p>

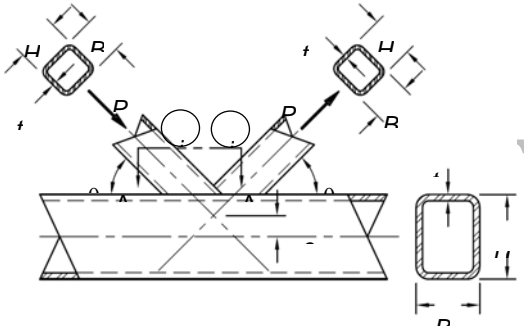
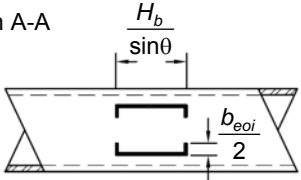


$$l_e = \frac{2(H_b - 1.2t_b)}{\sin\theta} + (B_b - 1.2t_b) \quad (\text{K4-6})$$

when  $50^\circ < \theta < 60^\circ$

Linear interpolation shall be used to determine  $l_e$

Table K4.1 (continued)

Overlapped K-Connections under Branch Axial Load	Overlapping Member Effective Weld Properties (all dimensions are for the overlapping branch, $i$ )
 <p>Note that the force arrows shown for overlapped K-connections may be reversed; <math>i</math> and <math>j</math> control member identification</p>	<p>when <math>25\% \leq O_v &lt; 50\%</math> :</p> $l_{e, \text{overlapping branch}} = \frac{2O_v}{50} \left[ \left( 1 - \frac{O_v}{100} \right) \left( \frac{H_{bi}}{\sin \theta_i} \right) + \frac{O_v}{100} \left( \frac{H_{bi}}{\sin(\theta_i + \theta_j)} \right) \right] + b_{eoi} + b_{eov} \quad (\text{K4-7})$ <p>when <math>50\% \leq O_v &lt; 80\%</math> :</p> $l_{e, \text{overlapping branch}} = 2 \left[ \left( 1 - \frac{O_v}{100} \right) \left( \frac{H_{bi}}{\sin \theta_i} \right) + \frac{O_v}{100} \left( \frac{H_{bi}}{\sin(\theta_i + \theta_j)} \right) \right] + b_{eoi} + b_{eov} \quad (\text{K4-8})$ <p>when <math>80\% \leq O_v \leq 100\%</math> :</p> $l_{e, \text{overlapping branch}} = 2 \left[ \left( 1 - \frac{O_v}{100} \right) \left( \frac{H_{bi}}{\sin \theta_i} \right) + \frac{O_v}{100} \left( \frac{H_{bi}}{\sin(\theta_i + \theta_j)} \right) \right] + B_{bi} + b_{eov} \quad (\text{K4-9})$
<p>Section A-A</p>  <p>Effective Weld</p>	$b_{eoi} = \frac{10}{B/t} \left( \frac{F_y t}{F_{yb} t_{bi}} \right) B_{bi} \leq B_{bi} \quad (\text{K2-20})$ $b_{eov} = \frac{10}{B_{bj}/t_{bj}} \left( \frac{F_{yb} t_{bj}}{F_{yb} t_{bi}} \right) B_{bi} \leq B_{bi} \quad (\text{K2-21})$
	<p>when <math>B_{bi}/B_b &gt; 0.85</math> or <math>\theta_2 &gt; 50^\circ</math>, <math>b_{eoi}</math> shall not exceed <math>2t</math>  and when <math>B_{bi}/B_{bj} &gt; 0.85</math> or <math>(180 - \theta_1 - \theta_2) &gt; 50^\circ</math>, <math>b_{eov}</math> shall not exceed <math>2t_{bj}</math></p> <p>Subscript <math>i</math> refers to the overlapping branch  Subscript <math>j</math> refers to the overlapped branch</p>
	<p>Overlapped Member Effective Weld Properties (all dimensions are for the overlapped branch, <math>j</math>)</p> $l_e = \frac{2H_b}{\sin \theta} + 2b_{eoj} \quad (\text{K4-10})$

$$b_{eoj} = \frac{10}{B/t} \left( \frac{F_y t}{F_{ybj} t_{bj}} \right) B_{bj} \leq B_{bj} \quad (\text{K4-11})$$

when  $B_{bj}/B > 0.85$  or  $\theta_2 > 50^\circ$ ,  $b_{eoj}$  shall not exceed  $2t$

**User Note:** The weld checks in Table K4.1 may be omitted if the welds are capable of developing the full strength of the branch member wall along its entire perimeter (or a plate along its entire length).

The approach used here to allow down-sizing of welds assumes a constant weld size around the full perimeter of the HSS branch, including all “non-effective” areas, except when the “hidden” weld under an overlapping branch is allowed to be omitted (discussed below). Special attention is required for equal width (or near-equal width) connections which combine partial penetration groove welds, along the matched edges of the connection, with fillet welds generally across the main member face.

A special case exists for branches with axial loads in overlapped K- connections. When this connection has been designed in accordance with Table K2.2 of this chapter, and the branch member component forces normal to the chord are 80% “balanced” (i.e. the branch member forces normal to the chord face differ by no more than 20%); the “hidden” weld under an overlapping branch may be omitted if:

- a. the remaining welds to the overlapped branch everywhere develop the full capacity of the overlapped branch member members walls, or
- b. the remaining welds to the overlapped branch are designed with a weld effective length of  $l_e = 2H_b/\sin(\theta)$  (i.e.  $b_{eoj}$  in K4-10 is taken as zero).

# CHAPTER L

## DESIGN FOR SERVICEABILITY

This chapter addresses serviceability design requirements.

The chapter is organized as follows:

- L1. General Provisions
- L2. Camber
- L3. Deflections
- L4. Drift
- L5. Vibration
- L6. Wind-Induced Motion
- L7. Expansion and Contraction
- L8. Connection Slip

### L1. GENERAL PROVISIONS

*Serviceability* is a state in which the function of a building, its appearance, maintainability, durability, and comfort of its occupants are preserved under normal usage. Limiting values of structural behavior for serviceability (such as maximum deflections and accelerations) shall be chosen with due regard to the intended function of the structure. Serviceability shall be evaluated using appropriate *load combinations* for the *serviceability limit states* identified.

**User Note:** *Serviceability limit states, service loads* and appropriate *load combinations* for *serviceability* requirements can be found in ASCE 7, Appendix C and Commentary to Appendix C. The performance requirements for *serviceability* in this chapter are consistent with those requirements. *Service loads*, as stipulated herein, are those that act on the structure at an arbitrary point in time and are not usually taken as the *nominal loads*.

### L2. CAMBER

Where *camber* is used to achieve proper position and location of the structure, the magnitude, direction, and location of camber shall be specified in the structural drawings.

### L3. DEFLECTIONS

Deflections in structural members and structural systems under appropriate *service load combinations* shall not impair the *serviceability* of the structure.

**User Note:** Conditions to be considered include levelness of floors, alignment of structural members, integrity of building finishes and other factors that affect the normal usage and function of the structure. For *composite* members, the additional deflections due to the shrinkage and creep of the concrete should be considered.

#### L4. DRIFT

*Drift* of a structure shall be evaluated under *service loads* to provide for *serviceability* of the structure, including the integrity of interior partitions and exterior *cladding*. *Drift* under strength *load combinations* shall not cause collision with adjacent structures or exceed the limiting values of such *drifts* that may be specified by the *applicable building code*.

#### L5. VIBRATION

The effect of vibration on the comfort of the occupants and the function of the structure shall be considered. The sources of vibration to be considered include pedestrian loading, vibrating machinery and others identified for the structure.

#### L6. WIND-INDUCED MOTION

The effect of wind-induced motion of buildings on the comfort of occupants shall be considered.

#### L7. EXPANSION AND CONTRACTION

The effects of thermal expansion and contraction of a building shall be considered. Damage to building *cladding* can cause water penetration and may lead to corrosion.

#### L8. CONNECTION SLIP

The effects of *connection slip* shall be included in the design where slip at bolted connections may cause deformations that impair the *serviceability* of the structure. Where appropriate, the connection shall be designed to preclude slip.

**User Note:** For the design of slip-critical connections, see Sections J3.8 and J3.9. For more information on connection slip, refer to the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts*.



## CHAPTER M

### FABRICATION, ERECTION AND QUALITY CONTROL

This chapter addresses requirements for shop drawings, fabrication, shop painting, erection and *quality control*.

The chapter is organized as follows:

- M1. Shop and Erection Drawings
- M2. Fabrication
- M3. Shop Painting
- M4. Erection

#### M1. SHOP AND ERECTION DRAWINGS

Shop and erection drawings are permitted to be prepared in stages. Shop drawings shall be prepared in advance of fabrication and give complete information necessary for the fabrication of the component parts of the structure, including the location, type and size of welds and bolts. Erection drawings shall be prepared in advance of erection and give information necessary for erection of the structure. Shop and erection drawings shall clearly distinguish between shop and field welds and bolts and shall clearly identify pretensioned and slip-critical high-strength bolted *connections*. Shop and erection drawings shall be made with due regard to speed and economy in fabrication and erection.

#### M2. FABRICATION

##### 1. Cambering, Curving and Straightening

Local application of heat or mechanical means is permitted to be used to introduce or correct *camber*, curvature and straightness. The temperature of heated areas, as measured by approved methods, shall not exceed 1,100 °F (593 °C) for A514/A514M and A852/A852M steel nor 1,200 °F (649 °C) for other steels.

##### 2. Thermal Cutting

*Thermally cut* edges shall meet the requirements of AWS D1.1, Sections 5.15.1.2, 5.15.4.3 and 5.15.4.4 with the exception that thermally cut free edges that will not be subject to fatigue shall be free of round-bottom *gouges* greater than  $\frac{3}{16}$  (5 mm) deep and sharp V-shaped notches. *Gouges* deeper than  $\frac{3}{16}$  in. (5 mm) and notches shall be removed by grinding or repaired by welding.

Reentrant corners shall be formed with a curved transition. The radius need not exceed that required to fit the connection. The surface resulting from two straight torch cuts meeting at a point is not considered to be curved. Discontinuous corners are permitted where the material on both sides of the discontinuous reentrant corner are connected to a mating piece to prevent deformation and associated stress concentration at the corner.

**User Note:** Reentrant corners with a radius of 1/2 to 3/8 in. (13 to 10 mm) are acceptable for statically loaded work. Where pieces need to fit tightly together, a discontinuous reentrant corner is acceptable if the pieces are connected close to the corner on both sides of the discontinuous corner. Slots in HSS for gussets may be made with semicircular ends, or with curved corners. Square ends are acceptable provided the edge of the gusset is welded to the HSS

Beam copes and weld access holes shall meet the geometrical requirements of Section J1.6. Beam copes and weld access holes in shapes that are to be galvanized shall be ground to bright metal. For shapes with a flange thickness not exceeding 2 in. (50 mm) the roughness of *thermally cut* surfaces of copes shall be no greater than a surface roughness value of 2,000  $\mu\text{in.}$  (50  $\mu\text{m}$ ) as defined in ASME B46.1. For beam copes and weld access holes in which the curved part of the access hole is thermally cut in ASTM A6/A6M hot-rolled shapes with a flange thickness exceeding 2 in. (50 mm) and welded built-up shapes with material thickness greater than 2 in. (50 mm), a preheat temperature of not less than 150 °F (66 °C) shall be applied prior to thermal cutting. The thermally cut surface of access holes in ASTM A6/A6M hot-rolled shapes with a flange thickness exceeding 2 in. (50 mm) and built-up shapes with a material thickness greater than 2 in. (50 mm) shall be ground and inspected for cracks using magnetic particle inspection in accordance with ASTM E709 or liquid penetrant in accordance with ASTM E165. Any crack is unacceptable regardless of size or location.

**User Note:** The AWS Surface Roughness Guide for Oxygen Cutting (AWS C4.1-77) sample 2 may be used as a guide for evaluating the surface roughness of copes in shapes with flanges not exceeding 2 in.

### 3. Planing of Edges

Planing or finishing of sheared or *thermally cut* edges of plates or shapes is not required unless specifically called for in the contract documents or included in a stipulated edge preparation for welding.

### 4. Welded Construction

The technique of welding, the workmanship, appearance and quality of welds, and the methods used in correcting nonconforming work shall

be in accordance with AWS D1.1 except as modified in Section J2.

## 5. Bolted Construction

Parts of bolted members shall be pinned or bolted and rigidly held together during assembly. Use of a drift pin in bolt holes during assembly shall not distort the metal or enlarge the holes. Poor matching of holes shall be cause for rejection.

Bolt holes shall comply with the provisions of the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, hereafter referred to as the RCSC *Specification*, Section 3.3 except that *thermally cut* holes are permitted with a surface roughness profile not exceeding 1,000  $\mu\text{in.}$  (25  $\mu\text{m}$ ) as defined in ASME B46.1. *Gouges* shall not exceed a depth of 1/16 in. (2 mm). Water jet cut holes are also permitted.

Fully inserted finger *shims*, with a total thickness of not more than 1/4 in. (6 mm) within a *joint*, are permitted in joints without changing the strength (based upon hole type) for the design of *connections*. The orientation of such shims is independent of the direction of application of the *load*.

The use of high-strength bolts shall conform to the requirements of the RCSC *Specification*, except as modified in Section J3.

## 6. Compression Joints

Compression *joints* that depend on contact bearing as part of the *splice* strength shall have the bearing surfaces of individual fabricated pieces prepared by milling, sawing or other suitable means.

## 7. Dimensional Tolerances

Dimensional tolerances shall be in accordance with Chapter 6 of the AISC *Code of Standard Practice for Steel Buildings and Bridges*, hereafter referred to as the AISC *Code of Standard Practice*.

## 8. Finish of Column Bases

*Column* bases and base plates shall be finished in accordance with the following requirements:

- (1) Steel bearing plates 2 in. (50 mm) or less in thickness are permitted without milling provided a satisfactory contact bearing is obtained. Steel bearing plates over 2 in. (50 mm) but not over 4 in. (100 mm) in thickness are permitted to be straightened by pressing or, if presses are not available, by milling for bearing surfaces (except as noted in subparagraphs 2 and 3 of this section), to obtain a satisfactory contact bearing. Steel bearing plates over 4

in. (100 mm) in thickness shall be milled for bearing surfaces (except as noted in subparagraphs 2 and 3 of this section).

(2) Bottom surfaces of bearing plates and *column* bases that are grouted to ensure full bearing contact on foundations need not be milled.

(3) Top surfaces of bearing plates need not be milled when complete-joint-penetration *groove welds* are provided between the *column* and the bearing plate.

## 9. Holes for Anchor Rods

Holes for anchor rods are permitted to be *thermally cut* in accordance with the provisions of Section M2.2.

## 10. Drain Holes

When water can collect inside *HSS* or box members, either during construction or during service, the member shall be sealed, provided with a drain hole at the base or protected by other suitable means.

## 11. Requirements for Galvanized Members

Members and parts to be galvanized shall be designed, detailed and fabricated to provide for flow and drainage of pickling fluids and zinc and to prevent pressure buildup in enclosed parts.

**User Note:** See *The Design of Products to be Hot-Dip Galvanized After Fabrication*, American Galvanizer's Association, and ASTM A123, A153, A384 and A780 for useful information on design and detailing of galvanized members. See Section M2.2 for requirements for copes of members to be galvanized.

## M3. SHOP PAINTING

### 1. General Requirements

Shop painting and surface preparation shall be in accordance with the provisions in Chapter 6 of the *AISC Code of Standard Practice*.

Shop paint is not required unless specified by the contract documents.

### 2. Inaccessible Surfaces

Except for contact surfaces, surfaces inaccessible after shop assembly shall be cleaned and painted prior to assembly, if required by the contract documents.

### 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*, Section 3.2.2(b).

### 4. Finished Surfaces

Machine-finished surfaces shall be protected against corrosion by a rust inhibitive coating that can be removed prior to erection, or which has characteristics that make removal prior to erection unnecessary.

### 5. Surfaces Adjacent to Field Welds

Unless otherwise specified in the design documents, surfaces within 2 in. (50 mm) of any field weld location shall be free of materials that would prevent proper welding or produce objectionable fumes during welding.

## M4. ERECTION

### 1. Alignment of Column Bases

*Column* bases shall be set level and to correct elevation with full bearing on concrete or masonry as defined in Chapter 7 of the AISC Code of Standard Practice.

### 2. Stability and Connections

The frame of steel skeleton buildings shall be carried up true and plumb within the limits defined in Chapter 7 of the AISC *Code of Standard Practice*. As erection progresses, the structure shall be secured to support dead, erection and other loads anticipated to occur during the period of erection. Temporary bracing shall be provided, in accordance with the requirements of the AISC *Code of Standard Practice*, wherever necessary to support the loads to which the structure may be subjected, including equipment and the operation of same. Such bracing shall be left in place as long as required for safety.

### 3. Alignment

No permanent bolting or welding shall be performed until the adjacent affected portions of the structure have been properly aligned.

### 4. Fit of Column Compression Joints and Base Plates

Lack of contact bearing not exceeding a gap of 1/16 in. (2 mm), regardless of the type of *splice* used (*partial-joint-penetration groove welded* or bolted), is permitted. If the gap exceeds 1/16 in. (2 mm), but is equal to or less than 1/4 in. (6 mm), and if an engineering investiga-

tion shows that sufficient contact area does not exist, the gap shall be packed out with nontapered steel *shims*. Shims need not be other than mild steel, regardless of the grade of the main material.

## 5. Field Welding

Surfaces in and adjacent to *joints* to be field welded shall be prepared as necessary to assure weld quality. This preparation shall include surface preparation necessary to correct for damage or contamination occurring subsequent to fabrication.

## 6. Field Painting

Responsibility for touch-up painting, cleaning and field painting shall be allocated in accordance with accepted local practices, and this allocation shall be set forth explicitly in the contract documents.

## CHAPTER N

### QUALITY CONTROL AND QUALITY ASSURANCE

This chapter addresses requirements for quality control, quality assurance and nondestructive testing for structural steel systems and steel elements of composite members for buildings and other structures.

**User Note:** This chapter does not address quality control or quality assurance for reinforcing steel, concrete materials or placement of concrete for composite members. This chapter does not address quality control or quality assurance for surface preparation or coatings.

The Chapter is organized as follows:

- N1. Scope
- N2. Fabricator's and Erector's Quality Control Program
- N3. Fabricator's and Erector's Documents
- N4. Inspection and Nondestructive Testing Personnel
- N5. Minimum Requirements for Inspection of Structural Steel Buildings
- N6. Minimum Requirements for Inspection of Composite Construction
- N7. Approved Fabricators and Erectors
- N8. Nonconforming Materials and Workmanship

#### N1. SCOPE

*Quality Control* (QC) shall be provided by the fabricator and erector as specified in this chapter. *Quality Assurance* (QA) shall be provided by others when required by the authority having jurisdiction (AHJ), applicable building code (ABC), purchaser, owner or engineer of record (EOR). Nondestructive Testing (NDT) shall be performed by the agency or firm responsible for *Quality Assurance*, except as permitted in accordance with Section N7.

#### N2. FABRICATOR'S AND ERECTOR'S QUALITY CONTROL PROGRAM

The fabricator and erector shall establish and maintain quality control procedures and perform inspections to ensure that the work is performed in accordance with this Specification and the *construction documents*.

Material identification procedures shall comply with the requirements of the AISC *Code of Standard Practice* Section 6.1, and shall be monitored by the fabricator's *Quality Control Inspector* (QCI).

The fabricator's QCI shall inspect the following as a minimum:

- (1) Welding, high-strength bolting, and details in accordance with Section N5
- (2) Cut and finished surfaces, in accordance with Section M2
- (3) Heating for straightening, cambering and curving, in accordance with Section M2.1
- (4) Tolerances for fabrication, in accordance with Section 6 of the AISC *Code of Standard Practice*

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The erector's QCI shall inspect the following as a minimum:

- (1) Welding, high-strength bolting, and details in accordance with Section N5
- (2) Steel deck and shear connector placement and attachment in accordance with Section N6, if a part of the erector's work
- (3) Field cut surfaces, in accordance with Section M2.2
- (4) Heating for straightening, in accordance with Section M2.1
- (5) Tolerances for erection, in accordance with Section 7 of the *AISC Code of Standard Practice*.

The fabricator and erector shall maintain records of the inspections performed.

### **N3. FABRICATOR'S AND ERECTOR'S DOCUMENTS**

#### **1. Submittals for Steel Construction**

The fabricator or erector shall submit the following documents for review by the *engineer of record* (EOR) or designee, prior to fabrication or erection, as applicable:

- (1) Shop drawings, unless shop drawings have been furnished by others
- (2) Erection drawings, unless erection drawings have been furnished by others

#### **2. Available Documents for Steel Construction**

The following documents shall be available for review by the EOR or designee prior to fabrication or erection, as applicable, unless otherwise required in the contract documents to be submitted:

- (1) For main structural steel elements, copies of material test reports in accordance with Section A3.1
- (2) For fasteners, copies of manufacturer's certifications in accordance with Section A3.3
- (3) For consumables for welding, copies of manufacturer's certifications in accordance with Section A3.5
- (4) For headed stud anchors, copies of manufacturer's certifications in accordance with Section A3.6
- (5) Manufacturer's product data sheets or catalog data for welding filler metals and fluxes to be used. The data sheets shall describe the product, limitations of use, recommended or typical welding parameters, and storage and exposure requirements, including baking, if applicable.
- (6) Welding procedure specifications (WPSs)
- (7) Procedure qualification records (PQRs) for WPSs that are not prequalified in accordance with AWS D1.1 or AWS D1.3, as applicable
- (8) Welder performance qualification records (WPQR) and continuity records
- (9) Fabricator's or erector's, as applicable, written quality control manual, that shall include, as a minimum:
  - (i) Material control procedures
  - (ii) QC Inspector qualifications
  - (iii) Inspection procedures
  - (iv) Nonconformance procedures



#### N4. INSPECTION AND NONDESTRUCTIVE TESTING PERSONNEL

##### 1. Quality Control Inspector Qualifications

*Quality control* (QC) welding inspection personnel shall be qualified to the satisfaction of the fabricator's or erector's QC program, as applicable, and in accordance with either of the following:

- (a) Associate welding inspectors (AWI) or higher as defined in AWS B5.1, *Standard for the Qualification of Welding Inspectors*, or
- (b) Qualified under the provisions of AWS D1.1/D1.1M subclause 6.1.4

QC bolting inspection shall be conducted by qualified personnel. The basis for qualification shall be documented training and experience in structural bolting inspection.

##### 2. Quality Assurance Inspector Qualifications

*Quality Assurance* (QA) welding inspectors shall be qualified to the satisfaction of the QA agency's written practice, and in accordance with either of the following:

- (a) Welding Inspectors (WIs), or Senior Welding Inspectors (SWIs), as defined in AWS B5.1, *Standard for the Qualification of Welding Inspectors*, except Associate Welding Inspectors (AWIs) are permitted to be used under the direct supervision of WIs, who are on site and available when weld inspection is being conducted, or
- (b) Qualified under the provisions of AWS D1.1/D1.1M subclause 6.1.4

QA bolting inspection shall be conducted by qualified personnel. The basis for qualification shall be documented training and experience in structural bolting inspection.

##### 3. NDT Personnel Qualifications

Nondestructive testing personnel shall be qualified in accordance with their employer's written practice, which shall meet or exceed the criteria of the American Society for Nondestructive Testing (ASNT) SNT-TC-1A, *Recommended Practice for the Qualification and Certification of Nondestructive Testing Personnel*, or ASNT CP-189, *Standard for the Qualification and Certification of Nondestructive Testing Personnel*, and AWS D1.1 *Structural Welding Code – Steel* clause 6.14.6.

#### N5. MINIMUM REQUIREMENTS FOR INSPECTION OF STRUCTURAL STEEL BUILDINGS

##### 1. Inspection Functions

Inspections shall be performed as *continuous inspection*, unless otherwise identified as permitted to be performed as *periodic inspection*.

##### 2. Quality Control

Inspection tasks shall be continuously performed by the person performing the fabrication or erection.

QC inspection tasks shall be performed as *continuous inspection* or *periodic inspection*, as designated, by the fabricator's or erector's *Quality Control Inspector* (QCI), as applicable, in accordance with Sections N5.5, N5.7 and N5.8.

Tasks in Tables N5.5-1 through N5.5-3 and Tables N5.7-1 through N5.7-3 listed for QC are those inspections performed by the QCI to ensure that the work is performed in accordance with the *construction documents*.

For QC inspection, the applicable construction documents are the *shop drawings* and the *erection drawings*. The QCI need not refer to the *design drawings* and *specifications*.

### 3. Quality Assurance

As much as practicable, *Quality Assurance* (QA) inspection of fabricated items shall be made at the fabricator's plant. The *Quality Assurance Inspector* (QAI) shall schedule this work to minimize interruption to the work of the fabricator.

QA inspection of the erected steel system shall be made at the project site. The QAI shall schedule this work to minimize interruption to the work of the erector.

QA inspection tasks shall be periodically or continuously performed, as designated, by the QAI, in accordance with Sections N5.5, N5.7 and N5.8.

Tasks in Tables N5.5-1 through N5.5-3 and N5.7-1 through N5.7-3 listed for QA are those inspections performed by the QAI to ensure that the work is performed in accordance with the *construction documents*.

Concurrent with the submittal of such reports to the AHJ, EOR or owner, the QA agency shall submit to the fabricator and erector:

- (1) Inspection reports
- (2) Nondestructive testing reports

### 4. Coordinated Inspection

Where a task is noted to be performed by both QC and QA, it is permitted to coordinate the inspection function between the QCI and QAI so that the inspection functions are performed by only one party. Where QA is to rely upon inspection functions performed by QC, the approval of the *engineer of record* and the *authority having jurisdiction* is required.

### 5. Inspection of Welding

Observation of welding operations and visual inspection of in-process and completed welds shall be the primary method to confirm that the materials, procedures, and workmanship are in conformance with the *construction documents*. For structural steel, all provisions of AWS D1.1/D1.1M *Structural Welding Code – Steel* for statically loaded

structures shall apply, except those items listed in Section J2 of this Specification.

Welding inspection shall be *continuous inspection*, except that *periodic inspection* is permitted for the following items, provided the welding consumables, welding procedure specifications and qualifications of welding personnel are verified prior to the start of the work, observations are made of the work in progress, and a visual inspection of all completed welds is made:

- (1) Single pass fillet welds
- (2) Welded studs when used for composite construction, in accordance with Section N6
- (3) Welding of stairs and railing systems and other steel, iron and metal items, as defined in Section 2.2 of the AISC *Code of Standard Practice*, made from standard structural shapes or hot-rolled steel plate.

**User Note:** The inspection of steel (open-web) joists and joist girders, tanks, pressure vessels, cables, cold-formed steel products, gage metal products is addressed in other standards.

As a minimum, welding inspection tasks shall be in accordance with Tables N5.5-1, N5.5-2 and N5.5-3. In these tables, the inspection tasks are as follows:

O – Observe these items on a random basis. Operations need not be delayed pending these inspections.

P – Perform these tasks prior to final acceptance of the item.

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**TABLE N5.5-1**

## Inspection Tasks Prior to Welding

<b>Inspection Tasks Prior to Welding</b>		
	<b>QC</b>	<b>QA</b>
Welding procedure specifications (WPSs) available	P	P
Manufacturer certifications for welding consumables available	P	P
Material identification (type/grade)	O	O
Welder identification system <sup>1</sup>	O	O
Fit-up of groove welds (including joint geometry) <ul style="list-style-type: none"> <li>- Joint preparation</li> <li>- Dimensions (alignment, root opening, root face, bevel)</li> <li>- Cleanliness (condition of steel surfaces)</li> <li>- Tacking (tack weld quality and location)</li> <li>- Backing type and fit (if applicable)</li> </ul>	P/O <sup>2</sup>	O
Configuration and finish of access holes	O	O
Fit-up of fillet welds <ul style="list-style-type: none"> <li>- Dimensions (alignment, gaps at root)</li> <li>- Cleanliness (condition of steel surfaces)</li> <li>- Tacking (tack weld quality and location)</li> </ul>	P/O <sup>2</sup>	O
<sup>1</sup> The fabricator or erector, as applicable, shall maintain a system by which a welder who has welded a joint or member can be identified. Stamps, if used, shall be the low-stress type. <sup>2</sup> Following performance of this inspection task for ten welds to be made by a given welder, with the welder demonstrating adequate understanding of requirements and possession of skills and tools to verify these items, the Perform designation of this task shall be reduced to Observe, and the welder shall perform this task. Should the inspector determine that the welder has discontinued adequate performance of this task, the task shall be returned to Perform until such time as the Inspector has reestablished adequate assurance that the welder will perform the inspection tasks listed.		

**TABLE N5.5-2**

## Inspection Tasks during Welding

<b>Inspection Tasks During Welding</b>	<b>QC</b>	<b>QA</b>
WPS followed <ul style="list-style-type: none"> <li>- Settings on welding equipment</li> <li>- Travel speed</li> <li>- Selected welding materials</li> <li>- Shielding gas type/flow rate</li> <li>- Preheat applied</li> <li>- Interpass temperature maintained (min/max.)</li> <li>- Proper position (F, V, H, OH)</li> </ul>	O	O
Use of qualified welders	O	O
Control and handling of welding consumables <ul style="list-style-type: none"> <li>- Packaging</li> <li>- Exposure control</li> </ul>	O	O
Environmental conditions <ul style="list-style-type: none"> <li>- Wind speed within limits</li> <li>- Precipitation and temperature</li> </ul>	O	O
Welding techniques <ul style="list-style-type: none"> <li>- Interpass and final cleaning</li> <li>- Each pass within profile limitations</li> <li>- Each pass meets quality requirements</li> </ul>	O	O
No welding over cracked tack welds	O	O

**TABLE N5.5-3**

## Inspection Tasks after Welding

<b>Inspection Tasks After Welding</b>	<b>QC</b>	<b>QA</b>
Welds cleaned	O	O
Size length, and location of welds	O	O
Welds meet visual acceptance criteria <ul style="list-style-type: none"> <li>- Crack prohibition</li> <li>- Weld/base-metal fusion</li> <li>- Crater cross-section</li> <li>- Weld profiles</li> <li>- Weld size</li> <li>- Undercut</li> <li>- Porosity</li> </ul>	P	P
Backing removed and weld tabs removed (if required)	P	P
Repair activities	P	P
Document acceptance or rejection of welded joint, member or structure, as applicable	P	P

## 6. Nondestructive Testing of Welded Joints

### 6a. Procedures

Ultrasonic testing (UT), magnetic particle testing (MT), and penetrant testing (PT), where required, shall be performed by QA in accordance with AWS D1.1. Acceptance criteria shall be AWS D1.1 for statically loaded structures, unless otherwise designated by the EOR.

### 6b. k-Area NDT

When welding of doubler plates, continuity plates, or stiffeners has been performed in the k-area, the web k-area within 3 in. (75 mm) of the weld shall be tested for cracks using MT.

### 6c. CJP Groove Weld NDT

For structures in Occupancy Category III or IV of Table 1-1, Occupancy Category of Buildings and Other Structures for Flood, Wind, Snow, Earthquake, and Ice Loads, of SEI/ASCE 7, *Minimum Design Loads for Buildings and Other Structures*, UT shall be performed on all CJP groove welds subject to transversely applied tension loading in butt, T- and corner joints, in materials 5/16 in. (8 mm) thick or greater. For structures in Occupancy Category II, UT shall be performed on 10% of CJP groove welds in butt, T- and corner joints subject to transversely applied tension loading, in materials 5/16 in. (8 mm) thick or greater.

**User Note:** For structures in Occupancy Category I, NDT of CJP groove welds is not required. For all structures in all Occupancy Categories, NDT of CJP groove welds in materials less than 5/16 in. (8 mm) thick is not required.

### 6d. Access Hole NDT

At welded splices and connections, thermally cut surfaces of access holes shall be tested using MT or PT, when the flange thickness exceeds 2 in. (50 mm) for rolled shapes, or when the web thickness exceeds 2 in. (50 mm) for built-up shapes. Any crack shall be deemed unacceptable regardless of size or location.

**User Note:** See Section M2.2.

### 6e. Welded Joints Subjected to Fatigue

When required by Appendix 3, Table A-3.1, welded joints requiring weld soundness to be established by radiographic or ultrasonic inspection shall be tested as prescribed. Reduction in the rate of UT is prohibited.

### 6f. Reduction of Rate of Ultrasonic Testing

The rate of UT is permitted to be reduced if approved by the EOR and the AHJ. Where the initial rate for UT is 100%, the NDT rate for an individual welder or welding operator is permitted to be reduced to 25%, provided the reject rate, the number of welds contain-

ing unacceptable defects divided by the number of welds completed, is demonstrated to be 5% or less of the welds tested for the welder or welding operator. A sampling of at least 40 completed welds for a job shall be made for such reduction evaluation. For evaluating the reject rate of continuous welds over 3 ft (1 m) in length where the effective throat is 1 in. (25 mm) or less, each 12 in. (300 mm) increment or fraction thereof shall be considered as one weld. For evaluating the reject rate on continuous welds over 3 ft (1 m) in length where the effective throat is greater than 1 in. (25 mm), each 6 in. (150 mm) of length or fraction thereof shall be considered one weld.

**6g. Increase in Rate of Ultrasonic Testing**

For structures in Occupancy Category II, where the initial rate for UT is 10 percent, the NDT rate for an individual welder or welding operator shall be increased to 100 percent should the reject rate, the number of welds containing unacceptable defects divided by the number of welds completed, exceeds 5 percent of the welds tested for the welder or welding operator. A sampling of at least 20 completed welds for a job shall be made prior to implementing such an increase. When the reject rate for the welder or welding operator, after a sampling of at least 40 completed welds, has fallen to 5 percent or less, the rate of UT shall be returned to 10%. For evaluating the reject rate of continuous welds over 3 ft (1 m) in length where the effective throat is 1 in. (25 mm) or less, each 12 in. (300 mm) increment or fraction thereof shall be considered as one weld. For evaluating the reject rate on continuous welds over 3 ft (1 m) in length where the effective throat is greater than 1 in. (25 mm), each 6 in. (150 mm) of length or fraction thereof shall be considered one weld.

**6h. Documentation**

All NDT performed shall be documented. For shop fabrication, the NDT report shall identify the tested weld by piece mark and location in the piece. For field work, the NDT report shall identify the tested weld by location in the structure, piece mark, and location in the piece.

When a weld is rejected on the basis of NDT, the NDT record shall indicate the location of the defect and the basis of rejection.

**7. Inspection of High-Strength Bolting**

Observation of bolting operations shall be the primary method used to confirm that the materials, procedures and workmanship incorporated in construction are in conformance with the *construction documents*. Inspection tasks prior to bolting in accordance with Table N5.7-1 shall be performed as *periodic inspection*.

- (1) For snug tight joints, inspection shall be performed as periodic inspection. For such joints, pre-installation verification testing as specified in Table N5.7-1 and monitoring of the installation procedures as specified in Table N5.7-2 are not applicable.
- (2) For pretensioned joints and slip-critical joints, when the installer is using the turn-of-nut method with matchmarking techniques, the direct-tension-indicator method, or the twist-off-type tension control bolt method, monitoring of bolt pretension-

2542 ing procedures as specified in Table N5.7-2 shall be performed as periodic in-  
2543 spection.  
2544

2545 (3) For pretensioned joints and slip-critical joints, when the installer is using the cali-  
2546 brated wrench method or the turn-of-nut method without matchmarking, monitor-  
2547 ing of bolt pretensioning procedures as specified in Table N5.7-2 shall be per-  
2548 formed as *continuous inspection*.  
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2550 As a minimum, bolting inspection tasks shall be in accordance with Tables N5.7-1, N5.7-  
2551 2 and N5.7-3. In these tables, the inspection tasks are as follows:  
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2553 O – Observe these items on a random basis. Operations need not be delayed pending  
2554 these inspections.

2555 P – Perform these tasks prior to final acceptance of the item.



**TABLE N5.7-1**

## Inspection Tasks Prior to Bolting

Inspection Tasks Prior to Bolting		
	QC	QA
Manufacturer's certifications available for fastener materials	O	P
Fasteners marked in accordance with ASTM requirements	O	O
Proper bolts selected for the joint detail	O	O
Proper bolting procedure selected for joint detail	O	O
Connecting elements, including the appropriate faying surface condition and hole preparation, if specified, meet applicable requirements	O	O
Pre-installation verification testing observed and documented by installation personnel for fastener assemblies and methods used	P	O
Proper storage provided for bolts, nuts, washers, and other fastener components	O	O

**TABLE N5.7-2**

## Inspection Tasks during Bolting

Inspection Tasks During Bolting		
	QC	QA
Fastener assemblies, of suitable condition, placed in all holes and washers (if required) are positioned as required	O	O
Joint brought to the snug tight condition prior to the pretensioning operation	O	O
Fastener component not turned by the wrench prevented from rotating	O	O
Bolts are pretensioned in accordance with a method approved by RCSC and progressing systematically from most rigid point toward free edges	O	O

**Table N5.7-3**

## Inspection Tasks after Bolting

Inspection Tasks After Bolting		
	QC	QA
Document acceptance or rejection of bolted connections, members or structure, as applicable	P	P

## 8. Other Inspection Tasks

The fabricator's QCI shall perform *periodic inspection* of the fabricated steel to verify compliance with the details shown on the *construction documents*, such as proper application of joint details at each connection. The erector's QCI shall perform periodic inspection of the erected steel frame to verify compliance with the details shown on the construction documents, such as bracing, stiffeners, member locations and proper application of joint details at each connection.

The QAI shall perform periodic inspection of the fabricated steel or erected steel frame, as appropriate, to verify compliance with the details shown on the construction documents, such as bracing, stiffeners, member locations and proper application of joint details at each connection.

The QAI shall perform *continuous inspection* during the placement of anchor rods and other embedments supporting structural steel for conformance with the construction documents. As a minimum, the diameter, grade, type and length of the anchor rod or embedded item, and the extent or depth of embedment into the concrete, shall be verified prior to placement to concrete.

## N6. MINIMUM REQUIREMENTS FOR INSPECTION OF COMPOSITE CONSTRUCTION

Inspection of structural steel and steel deck used in composite construction shall comply with the requirements this Chapter.

For welding of steel deck, observation of welding operations and visual inspection of in-process and completed welds shall be the primary method to confirm that the materials, procedures, and workmanship are in conformance with the *construction documents*. All provisions of AWS D1.1/D1.3M *Structural Welding Code – Sheet Steel* shall apply. Deck welding inspection shall be *periodic inspection* provided the welding consumables, welding procedure specifications and qualifications of welding personnel are verified prior to the start of the work, observations are made of the work in progress, and a visual inspection of all completed welds is made.

For those items for Quality Control (QC) in Table N6-1 that contain an Observe designation, the QC inspection shall be performed by the erector's *Quality Control Inspector* (QCI). In Table N6-1, the inspection tasks are as follows:

O – Observe these items on a random basis. Operations need not be delayed pending these inspections.

P – Perform these tasks prior to final acceptance of the item.

**TABLE N6-1**

Inspection of Composite Construction Prior to Concrete Placement

<b>Inspection of Composite Construction Prior to Concrete Placement</b>	<b>QC</b>	<b>QA</b>
Placement and installation of steel deck	P	P
Placement and installation of stud shear connectors	P	P

**N7. APPROVED FABRICATORS AND ERECTORS**

*Quality Assurance* (QA) inspections, except nondestructive testing (NDT), may be waived when the work is performed in a fabricating shop or by an erector approved by the *authority having jurisdiction* (AHJ) to perform the work without QA. NDT of welds completed in an approved fabricator's shop may be performed by that fabricator when approved by the AHJ. When the fabricator performs the NDT, the QA agency shall review the fabricator's NDT reports.

At completion of fabrication, the approved fabricator shall submit a certificate of compliance to the AHJ stating that the materials supplied and work performed by the fabricator are in accordance with the construction documents. At completion of erection, the approved erector shall submit a certificate of compliance to the AHJ stating that the materials supplied and work performed by the erector are in accordance with the construction documents.

**N8. NONCONFORMING MATERIAL AND WORKMANSHIP**

Identification and rejection of material or workmanship that is not in conformance with the *construction documents* shall be permitted at any time during the progress of the work. Nonconforming material and workmanship shall be brought to the immediate attention of the fabricator or erector, as applicable.

Nonconforming material or workmanship shall be brought into conformance, or made suitable for its intended purpose as determined by the *engineer of record*.

Concurrent with the submittal of such reports to the AHJ, EOR or owner, the QA agency shall submit to the fabricator and erector:

- (1) Nonconformance reports
- (2) Reports of repair, replacement, or acceptance of nonconforming items

## APPENDIX 1

### DESIGN BY INELASTIC ANALYSIS

This appendix addresses design by inelastic analysis, in which consideration of the redistribution of member and connection forces and moments as a result of localized yielding is permitted.

The appendix is organized as follows:

- 1.1. General Requirements
- 1.2. Ductility Requirements
- 1.3. Analysis Requirements

#### 1.1. GENERAL REQUIREMENTS

Design by *inelastic analysis* shall be conducted in accordance with Section B3.3, *Load and Resistance Factor Design* (LRFD). The *design strength* of the structural system and its members and connections shall equal or exceed the *required strength* as determined by the inelastic analysis. The provisions of this Appendix do not apply to seismic design.

The inelastic analysis shall take into account: (1) flexural, shear, and axial member deformations, and all other component and *connection* deformations that contribute to the displacements of the structure; (2) *second-order effects* (including  $P-\Delta$  and  $P-\delta$  effects); (3) geometric imperfections; (4) stiffness reductions due to inelasticity, including the effect of residual stresses and partial yielding of the cross section; and (5) uncertainty in system, member, and connection strength and stiffness.

Strength limit states detected by an inelastic analysis that incorporates all of the above requirements are not subject to the corresponding provisions of the Specification when a comparable or higher level of reliability is provided by the analysis. It must be demonstrated that the capabilities of the inelastic analysis are adequate for waiving Specification provisions. In all cases, the *plastic strength* of the member cross section, calculated using the specified minimum yield stress, shall not be exceeded. Connections shall meet the requirements of Section B3.6. Connections at locations undergoing significant plastic deformation (flexural or axial) shall be designed based on a required strength equal to the strength of the yielding member calculated using the expected yield strength. Strength limit states not detected by the inelastic analysis shall be checked using the relevant provisions of Chapters D, E, F, G, H, I, J and K, considering the expected yield strength of the yielding elements.

Furthermore, members and connections subjected to inelastic deformations shall be shown to have adequate ductility consistent with the intended behavior of the structural system. Force redistribution due to rupture of a member or connection is not permitted.

Any method that uses inelastic analysis to proportion members and connections to satisfy these general requirements is permitted. A design method based on inelastic analysis that meets the above *strength requirements*, the *ductility requirements* of Section 1.2 and the *analysis requirements* of Section 1.3 satisfies these general requirements.

## 1.2. DUCTILITY REQUIREMENTS

Members and connections with elements yielding shall be proportioned such that all inelastic deformation demands are less than or equal to their inelastic deformation capacities. In lieu of explicitly ensuring that the inelastic deformation demands are less than or equal to their inelastic deformation capacities, the following requirements shall be satisfied for members subjected to plastic hinging:

### 1. Material

The *specified minimum yield strength*,  $F_y$ , of members subject to plastic hinging shall not exceed 65 ksi (450 MPa).

### 2. Cross-Section

The cross-section of members at plastic hinge locations shall be doubly-symmetric with width-thickness ratios of their compression elements not exceeding  $\lambda_{pd}$ , where  $\lambda_{pd}$  is equal to  $\lambda_p$  from Table B4.1 except as modified below:

- (a) For the width-thickness ratio ( $h/t_w$ ) of webs of I-shaped sections, rectangular HSS, and box-shaped sections subjected to combined flexure and compression

- (i) When  $P_u/\phi_c P_y \leq 0.125$

$$\lambda_{pd} = 3.76 \sqrt{\frac{E}{F_y}} \left( 1 - \frac{2.75 P_u}{\phi_c P_y} \right) \quad (\text{A-1-1})$$

- (ii) When  $P_u/\phi_c P_y > 0.125$

$$\lambda_{pd} = 1.12 \sqrt{\frac{E}{F_y}} \left( 2.33 - \frac{P_u}{\phi_c P_y} \right) \geq 1.49 \sqrt{\frac{E}{F_y}} \quad (\text{A-1-2})$$

where

$h$  = as defined in Section B4.1, in. (mm)

$t_w$  = web thickness, in. (mm)  
 $P_u$  = required axial strength in compression, kips (N)  
 $P_y$  = member yield strength  $F_y A_g$ , kips (N)  
 $\phi_c$  = resistance factor for compression = 0.90

- (b) For the width-thickness ratio ( $b/t$ ) of flanges of rectangular *HSS* and box-shaped sections, and for flange *cover plates*, and *diaphragm plates* between lines of *fasteners* or welds

$$\lambda_{pd} = 0.94 \sqrt{E / F_y} \quad (\text{A-1-3})$$

where

$b$  = as defined in Section B4.1, in. (mm)  
 $t$  = as defined in Section B4.1, in. (mm)

- (c) For the diameter-to-thickness ratio ( $D/t$ ) of circular *HSS* in flexure

$$\lambda_{pd} = 0.045E/F_y \quad (\text{A-1-4})$$

where

$D$  = outside diameter of circular *HSS* in. (mm)

### 3. Unbraced Length

In prismatic member segments that contain plastic hinges, the laterally *unbraced length* about the cross-section minor axis shall not exceed any of the  $L_{pd}$  limits determined as follows:

- (a) For I-shaped members bent about their major-axis:

$$L_{pd} = \left[ 0.12 - 0.076 \frac{M_1}{M_2} \right] \frac{E}{F_y} r_y \quad (\text{A-1-5})$$

where

$M_2$  = absolute value of the larger major-axis bending moment at the end of the unbraced segment, kip-in. (N-mm)

$$M_1 = M_2 - 2M_{mid} \leq M_2$$

$$M_1 \geq M_0$$

$M_{mid}$  = major-axis bending moment at the middle of the unbraced segment, kip-in. (N-mm)

$M_0$  = smaller major-axis bending moment at the end of the unbraced segment, kip-in. (N-mm)

$M_{mid}$  and  $M_0$  are positive when member is bent in single curvature and negative when bent in double curvature.

$r_y$  = radius of gyration about minor axis, in. (mm)

- (b) For solid rectangular bars and for rectangular *HSS* and box-shaped members bent about their major-axis

$$L_{pd} = \left[ 0.17 - 0.10 \frac{M_1}{M_2} \right] \frac{E}{F_y} r_y \geq 0.10 \frac{E}{F_y} r_y \quad (\text{A-1-6})$$

- (c) For all types of members subjected to axial compression

$$L_{pd} = 4.71 r_y \sqrt{E/F_y} \quad (\text{A-1-7})$$

For member segments subjected to axial compression and containing plastic hinges, the *unbraced length* about the cross-section major-axis shall not exceed  $4.71 r_x \sqrt{E/F_y}$

There is no  $L_{pd}$  limit for member segments containing plastic hinges in the following cases:

- Members with circular or square cross sections subjected only to flexure or to combined flexure and tension.
- Members subjected only to flexure about their minor axis or combined tension and flexure about their minor axis.
- Members subjected only to tension.

#### 4. Axial Force

To assure adequate ductility in compression members with plastic hinges, the calculated axial compressive force,  $P_u$ , shall be limited to  $0.75 F_y A_g$ .

### 1.3. ANALYSIS REQUIREMENTS

The *structural analysis* shall satisfy the general requirements of Section 1.1. These requirements are permitted to be satisfied by a second-order inelastic analysis that meets the following requirements.

A *first-order inelastic analysis* or *plastic mechanism* analysis that neglects the effects of geometric imperfections, partial yielding, and residual stresses is permitted for continuous beams not subjected to axial compression.

**User Note:** Refer to the Commentary for guidance in conducting a traditional plastic analysis and design in conformance with these provisions.

## 1. Material Properties and Yield Criteria

The specified minimum yield stress  $F_y$  and the stiffness of all steel members and connections shall be reduced by a factor of 0.9 for the analysis, except as noted below in Section 1.3.3.

The influence of axial force, major-axis bending moment, and minor-axis bending moment shall be included in the calculation of the inelastic response.

The plastic strength of the member cross section shall be represented in the analysis either by an elastic-perfectly-plastic yield criterion expressed in terms of the axial force, major-axis bending moment, and minor-axis bending moment, or by explicit modeling of the material stress-strain response as elastic-perfectly-plastic.

## 2. Geometric Imperfections

The analysis shall include the effects of initial geometric imperfections. This shall be done by explicitly modeling the imperfections as specified in Section C2.2a or by the application of equivalent notional loads as specified in Section C2.2b.

## 3. Residual Stress and Partial Yielding Effects

The analysis shall include the influence of residual stresses and partial yielding. This shall be done by explicitly modeling these effects in the analysis or by reducing the stiffness of all structural components as specified in Section C2.3.

If the provisions of Section C2.3 are used, then:

- (1) The 0.9 stiffness reduction factor specified in Section 1.3 shall be replaced by the reduction of the elastic modulus  $E$  by 0.8 as specified in Section C2.3, and
- (2) The elastic-perfectly-plastic yield criterion, expressed in terms of the axial force, major-axis bending moment and minor-axis bending moment, shall satisfy the cross-section strength limit defined by Equations H1-1a and H1-1b using  $P_c = 0.9P_y$ ,  $M_{cx} = 0.9M_{px}$  and  $M_{cy} = 0.9M_{py}$ .



## APPENDIX 2

### DESIGN FOR PONDING

This Appendix provides methods for determining whether a roof system has adequate strength and stiffness to resist ponding.

The appendix is organized as follows:

2.1. Simplified Design for Ponding

2.2. Improved Design for Ponding

#### 2.1. SIMPLIFIED DESIGN FOR PONDING

The roof system shall be considered stable for *ponding* and no further investigation is needed if both of the following two conditions are met:

$$C_p + 0.9C_s \leq 0.25 \quad (\text{A-2-1})$$

$$I_d \geq 25(S^4)10^{-6} \quad (\text{A-2-2})$$

$$(\text{S.I.: } I_d \geq 3\,940\,S^4) \quad (\text{A-2-2M})$$

where

$$C_p = \frac{32L_s L_p^4}{10^7 I_p} \quad (\text{A-2-3})$$

$$\left( \text{S.I.: } C_p = \frac{504L_s L_p^4}{I_p} \right) \quad (\text{A-2-3M})$$

$$C_s = \frac{32S L_s^4}{10^7 I_s} \quad (\text{A-2-4})$$

$$\left( \text{S.I.: } C_s = \frac{504S L_s^4}{I_s} \right) \quad (\text{A-2-4M})$$

$L_p$  = column spacing in direction of girder (length of primary members), ft (m)

$L_s$  = column spacing perpendicular to direction of girder (length of secondary members), ft (m)

$S$  = spacing of secondary members, ft (m)

$I_p$  = moment of inertia of primary members, in.<sup>4</sup> (mm<sup>4</sup>)

$I_s$  = moment of inertia of secondary members, in.<sup>4</sup> (mm<sup>4</sup>)

$I_d$  = moment of inertia of the steel deck supported on secondary members, in.<sup>4</sup> per ft (mm<sup>4</sup> per m)

For trusses and steel joists, the calculation of the moments of inertia  $I_p$  and  $I_s$  shall include the effects of web member strain when used in the above equation.

**User Note:** When the moment of inertia is calculated using only the truss or joist chord areas, the reduction in the moment of inertia due to web strain can typically be taken as 15%.

A steel deck shall be considered a secondary member when it is directly supported by the primary members.

## 2.2. IMPROVED DESIGN FOR PONDING

The provisions given below are to be used when a more accurate evaluation of framing *stiffness* is needed than that given by Equations A-2-1 and A-2-2.

Define the stress indexes

$$U_p = \left( \frac{0.8F_y - f_o}{f_o} \right)_p \quad \text{for the primary member} \quad (\text{A-2-5})$$

$$U_s = \left( \frac{0.8F_y - f_o}{f_o} \right)_s \quad \text{for the secondary member} \quad (\text{A-2-6})$$

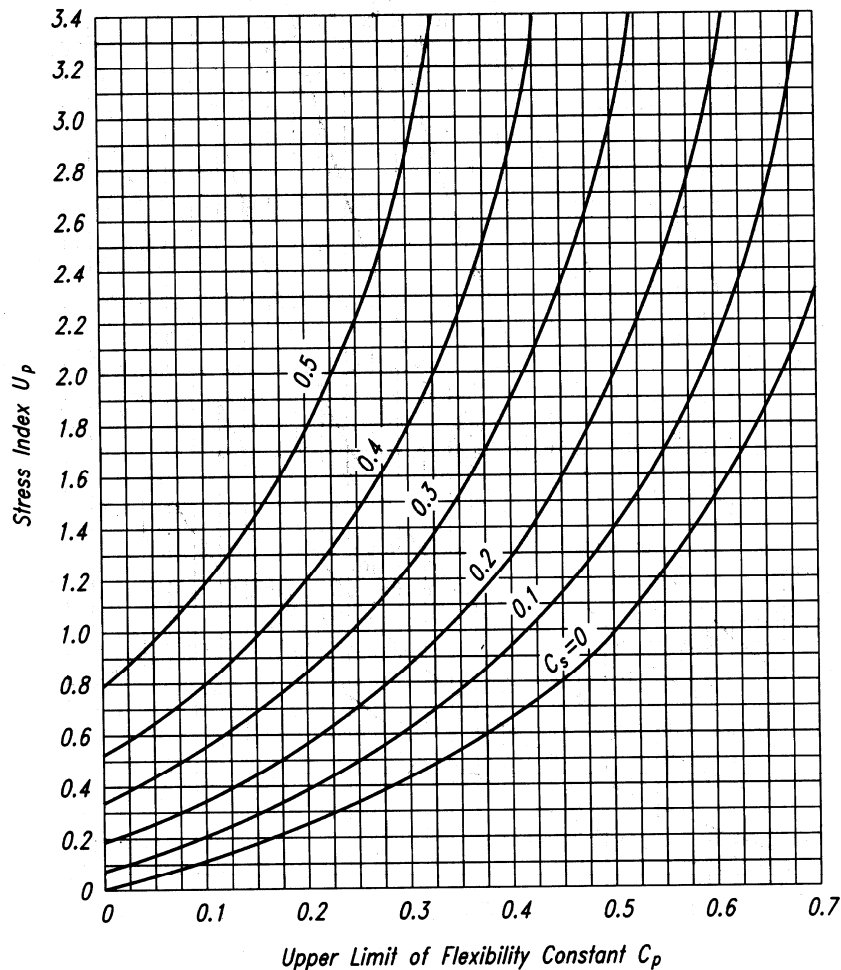
where

$f_o$  = stress due to  $D + R$  ( $D$  = nominal dead load,  $R$  = nominal load due to rainwater or snow exclusive of the *ponding* contribution) ksi (MPa)

For roof framing consisting of primary and secondary members, evaluate the combined stiffness as follows. Enter Figure A-2-1 at the level of the computed *stress* index  $U_p$  determined for the primary *beam*; move horizontally to the computed  $C_s$  value of the secondary *beams* and then downward to the abscissa scale. The combined *stiffness* of the primary and secondary framing is sufficient to prevent *ponding* if the flexibility 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.

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A similar procedure must be followed using Figure A-2-2.



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Fig. A-2-1. Limiting flexibility coefficient for the primary systems.

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For roof framing consisting of a series of equally-spaced wall bearing beams, evaluate the stiffness as follows. The beams are considered as secondary members supported on an infinitely stiff primary member. For this case, enter Figure A-2-2 with the computed *stress* index  $U_s$ . The limiting value of  $C_s$  is determined by the intercept of a horizontal line representing the  $U_s$  value and the curve for  $C_p = 0$ .

**User Note:** The *ponding* deflection contributed by a metal deck is usually such a small part of the total *ponding* deflection of a roof panel that it is sufficient merely to limit its moment of inertia (per foot (meter) of width normal to its span) to 0.000 025 (3 940) times the fourth power of its span length.

Evaluate the *stability* against *ponding* of a roof consisting of a metal roof deck of relatively slender depth-span ratio, spanning between beams supported directly on columns, as follows. Use Figure A-2-1 or

2993 A-2-2, using as  $C_s$  the flexibility constant for a one-foot (one-meter)  
 2994 width of the roof deck ( $S = 1.0$ ).  
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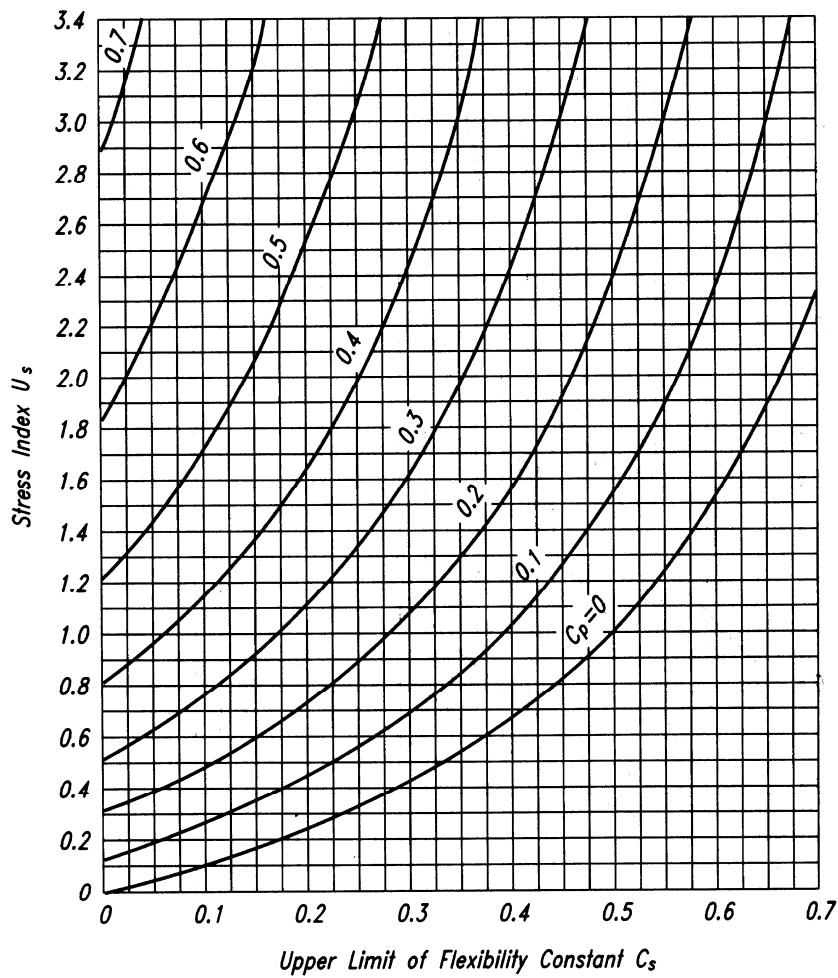


Fig. A-2-2. Limiting flexibility coefficient for the secondary systems.

## APPENDIX 3

### DESIGN FOR FATIGUE

This appendix applies to members and *connections* subject to high cycle loading within the elastic range of stresses of frequency and magnitude sufficient to initiate cracking and progressive failure, which defines the *limit state* of *fatigue*.

**User Note:** See AISC *Seismic Provisions for Structural Steel Buildings* for structures subject to seismic loads.

The appendix is organized as follows:

- 3.1. General Provisions
- 3.2. Calculation of Maximum Stresses and Stress Ranges
- 3.3. Design Stress Range
- 3.4. Bolts and Threaded Parts
- 3.5. Special Fabrication and Erection Requirements

#### 3.1. GENERAL PROVISIONS

The provisions of this Appendix apply to stresses calculated on the basis of *service loads*. The maximum permitted *stress* due to service 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 groove welds, the maximum *design stress range* calculated by Equation A-3-1 applies only to welds that have been ultrasonically or radiographically tested and meet the acceptance requirements of Section 6.12.2 or 6.13.2 of AWS D1.1.

No evaluation of *fatigue* resistance is required if the live load stress range is less than the threshold stress range,  $F_{TH}$ . See Table A-3.1.

No evaluation of fatigue resistance of members consisting of shapes or plate is required if the number of cycles of application of live load is less than 20,000. No evaluation of fatigue resistance of members consisting of HSS in building-type structures subject to code mandated wind loads is required.

The cyclic load resistance determined by the provisions of this Appendix is applicable to structures with suitable corrosion protection or subject only to mildly corrosive atmospheres, such as normal atmospheric conditions.

The cyclic load resistance determined by the provisions of this Appendix is applicable only to structures subject to temperatures not exceeding 300 °F (150 °C).

The *engineer of record* shall provide either complete details including weld sizes or shall specify the planned cycle life and the maximum range of moments, shears and reactions for the *connections*.

### 3.2. CALCULATION OF MAXIMUM STRESSES AND STRESS RANGES

Calculated stresses shall be based upon *elastic analysis*. Stresses shall not be amplified by *stress concentration* factors for geometrical discontinuities.

For bolts and threaded rods subject to axial tension, the calculated stresses shall include the effects of *prying action*, if any. In the case of axial stress combined with bending, the maximum stresses, of each kind, shall be those determined for concurrent arrangements of the applied *load*.

For members having symmetric cross sections, the *fasteners* and welds shall be arranged symmetrically about the axis of the member, or the total stresses including those due to eccentricity shall be included in the calculation of the *stress* range.

For axially loaded angle members where the center of gravity of the connecting welds lies between the line of the center of gravity of the angle cross section and the center of the connected leg, the effects of eccentricity shall be ignored. If the center of gravity of the connecting welds lies outside this zone, the total stresses, including those due to *joint eccentricity*, shall be included in the calculation of *stress* range.

### 3.3. DESIGN STRESS RANGE

The range of *stress* at *service loads* shall not exceed the *design stress range* computed as follows.

- (a) For stress categories A, B, B', C, D, E and E' the design stress range,  $F_{SR}$ , shall be determined by Equation A-3-1 or A-3-1M, as follows:

$$F_{SR} = \left( \frac{C_f}{n_{SR}} \right)^{0.333} \geq F_{TH} \quad (\text{A-3-1})$$

$$F_{SR} = \left( \frac{C_f \times 329}{n_{SR}} \right)^{0.333} \geq F_{TH} \quad (\text{S.I.}) \quad (\text{A-3-1M})$$

where

$C_f$  = constant from Table A-3.1 for the category

$F_{SR}$  = service stress range, ksi (MPa)

$F_{TH}$  = threshold *fatigue stress* range, maximum *stress* range for indefinite design life from Table A-3.1, ksi (MPa)

$n_{SR}$  = number of stress range fluctuations in design life

= number of stress range fluctuations per day  $\times$  365  $\times$  years of design life

- (b) For stress category F, the design stress range,  $F_{SR}$ , shall be determined by Equation A-3-2 or A-3-2M as follows:

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$$F_{SR} = \left( \frac{C_f}{n_{SR}} \right)^{0.167} \geq F_{TH} \quad (\text{A-3-2})$$

$$F_{SR} = \left( \frac{C_f \times 11 \times 10^4}{n_{SR}} \right)^{0.167} \geq F_{TH} \quad (\text{S.I.}) \quad (\text{A-3-2M})$$

(c) For tension-loaded plate elements connected at their end by cruciform, T, or corner details with *complete-joint-penetration (CJP) groove welds* or *partial-joint-penetration (PJP) groove welds*, *fillet welds*, or combinations of the preceding, transverse to the direction of stress, the design stress range on the cross section of the tension-loaded plate element at the toe of the weld shall be determined as follows:

(i) Based upon crack initiation from the toe of the weld on the tension loaded plate element the design stress range,  $F_{SR}$ , shall be determined by Equation A-3-3 or A-3-3M, for stress category C as follows:

$$F_{SR} = \left( \frac{44 \times 10^8}{n_{SR}} \right)^{0.333} \geq 10 \quad (\text{A-3-3})$$

$$F_{SR} = \left( \frac{14.4 \times 10^{11}}{n_{SR}} \right)^{0.333} \geq 68.9 \quad (\text{S.I.}) \quad (\text{A-3-3M})$$

(ii) Based upon crack initiation from the root of the weld the design stress range,  $F_{SR}$ , on the tension loaded plate element using transverse PJP groove welds, with or without reinforcing or contouring fillet welds, the design stress range on the cross section at the toe of the weld shall be determined by Equation A-3-4 or A-3-4M, stress category C' as follows:

$$F_{SR} = R_{PJP} \left( \frac{44 \times 10^8}{n_{SR}} \right)^{0.333} \quad (\text{A-3-4})$$

$$F_{SR} = R_{PJP} \left( \frac{14.4 \times 10^{11}}{n_{SR}} \right)^{0.333} \quad (\text{S.I.}) \quad (\text{A-3-4M})$$

where

$R_{PJP}$  is the reduction factor for reinforced or nonreinforced transverse PJP groove welds determined as follows:



$$R_{PJP} = \left( \frac{0.65 - 0.59 \left( \frac{2a}{t_p} \right) + 0.72 \left( \frac{w}{t_p} \right)}{t_p^{0.167}} \right) \leq 1.0$$

$$R_{PJP} = \left( \frac{1.12 - 1.01 \left( \frac{2a}{t_p} \right) + 1.24 \left( \frac{w}{t_p} \right)}{t_p^{0.167}} \right) \leq 1.0 \quad (\text{S.I.})$$

If  $R_{PJP} = 1.0$ , use stress category C.

- $2a$  = the length of the nonwelded root face in the direction of the thickness of the tension-loaded plate, in. (mm)  
 $w$  = the leg size of the reinforcing or contouring fillet, if any, in the direction of the thickness of the tension-loaded plate, in. (mm)  
 $t_p$  = thickness of tension loaded plate, in. (mm)

- (iii) Based upon crack initiation from the roots of a pair of transverse fillet welds on opposite sides of the tension loaded plate element the design stress range,  $F_{SR}$ , on the cross section at the toe of the welds shall be determined by Equation A-3-5 or A-3-5M, stress category C'' as follows:

$$F_{SR} = R_{FIL} \left( \frac{44 \times 10^8}{n_{SR}} \right)^{0.333} \quad (\text{A-3-5})$$

$$F_{SR} = R_{FIL} \left( \frac{14.4 \times 10^{11}}{n_{SR}} \right)^{0.333} \quad (\text{S.I.}) \quad (\text{A-3-5M})$$

where

$R_{FIL}$  is the reduction factor for joints using a pair of transverse fillet welds only.

$$R_{FIL} = \left( \frac{0.06 + 0.72 (w/t_p)}{t_p^{0.167}} \right) \leq 1.0$$

$$R_{FIL} = \left( \frac{0.10 + 1.24 (w/t_p)}{t_p^{0.167}} \right) \leq 1.0 \quad (\text{S.I.})$$

If  $R_{FIL} = 1.0$ , use stress category C.

### 3.4. BOLTS AND THREADED PARTS

The range of *stress* at *service loads* shall not exceed the stress range computed as follows.

- (a) For mechanically fastened *connections* loaded in shear, the maximum range of stress in the connected material at service loads shall not exceed the *design stress range* computed using Equation A-3-1 where  $C_f$  and  $F_{TH}$  are taken from Section 2 of Table A-3.1.
- (b) For high-strength bolts, common bolts and threaded anchor rods with cut, ground or rolled threads, the maximum range of tensile stress on the net tensile area from applied axial *load* and moment plus load due to *prying action* shall not exceed the design stress range computed using Equation A-3-6 or A-3-6M (stress category G). The net tensile area is given by Equation A-3-7 or A-3-7M.

$$F_{SR} = \left( \frac{3.9 \times 10^8}{n_{SR}} \right)^{0.333} \geq 7 \quad (\text{A-3-6})$$

$$F_{SR} = \left( \frac{1.28 \times 10^{11}}{n_{SR}} \right)^{0.333} \geq 48 \quad (\text{S.I.}) \quad (\text{A-3-6M})$$

$$A_t = \frac{\pi}{4} \left( d_b - \frac{0.9743}{n} \right)^2 \quad (\text{A-3-7})$$

$$A_t = \frac{\pi}{4} (d_b - 0.9382p)^2 \quad (\text{S.I.}) \quad (\text{A-3-7M})$$

where

$p$  = *pitch*, in. per thread (mm per thread)

$d_b$  = the nominal diameter (body or shank diameter), in. (mm)

$n$  = threads per in. (threads per mm)

For *joints* in which the material within the *grip* is not limited to steel or joints which are not tensioned to the requirements of Table J3.1 or J3.1M, all axial load and moment applied to the joint plus effects of any *prying action* shall be assumed to be carried exclusively by the bolts or rods.

For joints in which the material within the *grip* is limited to steel and which are pretensioned to the requirements of Table J3.1 or J3.1M, an analysis of the relative *stiffness* of the connected parts and bolts shall be permitted to be used to determine the tensile *stress* range in the pretensioned bolts due to the total service live load and moment plus effects of any prying action. Alternatively, the stress range in the bolts shall be assumed to be equal to the stress on the net tensile area due to 20 percent of the absolute value of the service load axial load and moment from dead, live and other loads.

### 3.5. SPECIAL FABRICATION AND ERECTION REQUIREMENTS

Longitudinal backing bars are permitted to remain in place, and if used, shall be continuous. If splicing is necessary for long *joints*, the bar shall be joined with complete penetration butt joints and the reinforcement ground prior to

assembly in the joint. Longitudinal backing, if left in place, shall be attached with continuous fillet welds.

In transverse joints subject to tension, backing bars, if used, shall be removed and the joint back gouged and welded.

In transverse complete-joint-penetration T and corner joints, a reinforcing *fillet weld*, not less than  $\frac{1}{4}$  in. (6 mm) in size shall be added at reentrant corners.

The surface roughness of thermally cut edges subject to cyclic *stress* ranges, that include tension, shall not exceed 1,000  $\mu\text{in.}$  (25  $\mu\text{m}$ ), where ASME B46.1 is the reference standard.

**User Note:** AWS C4.1 Sample 3 may be used to evaluate compliance with this requirement.

Reentrant corners at cuts, *cope*s and weld access holes shall form a radius of not less than  $\frac{3}{8}$  in. (10 mm) by predrilling or subpunching and reaming a hole, or by thermal cutting to form the radius of the cut. If the radius portion is formed by thermal cutting, the cut surface shall be ground to a bright metal surface.

For transverse butt joints in regions of tensile stress, weld tabs shall be used to provide for cascading the weld termination outside the finished joint. End dams shall not be used. Run-off tabs shall be removed and the end of the weld finished flush with the edge of the member.

See Section J2.2b for requirements for *end returns* on certain fillet welds subject to cyclic *service loading*.

**TABLE A-3.1**  
**Fatigue Design Parameters**

Description	Stress Category	Constant $C_f$	Threshold $F_{TH}$ ksi (MPa)	Potential Crack Initiation Point
<b>SECTION 1 – PLAIN MATERIAL AWAY FROM ANY WELDING</b>				
1.1 Base metal, except non-coated weathering steel, with rolled or cleaned surface. Flame-cut edges with surface roughness value of 1,000 $\mu$ in. (25 $\mu$ m) or less, but without reentrant corners.	A	$250 \times 10^8$	24 (165)	Away from all welds or structural connections
1.2 Non-coated weathering steel base metal with rolled or cleaned surface. Flame-cut edges with surface roughness value of 1,000 $\mu$ in. (25 $\mu$ m) or less, but without reentrant corners.	B	$120 \times 10^8$	16 (110)	Away from all welds or structural connections
1.3 Member with drilled or reamed holes. Member with re-entrant corners at copes, cuts, block-outs or other geometrical discontinuities made to requirements of Appendix 3.5, except weld access holes.	B	$120 \times 10^8$	16 (110)	At any external edge or at hole perimeter
1.4 Rolled cross sections with weld access holes made to requirements of Section J1.6 and Appendix 3.5. Members with drilled or reamed holes containing bolts for attachment of light bracing where there is a small longitudinal component of brace force.	C	$44 \times 10^8$	10 (69)	At reentrant corner of weld access hole or at any small hole (may contain bolt for minor connections)
<b>SECTION 2 – CONNECTED MATERIAL IN MECHANICALLY FASTENED JOINTS</b>				
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	$120 \times 10^8$	16 (110)	Through gross section near hole
2.2 Base metal at net section of high-strength bolted joints, designed on the basis of bearing resistance, but fabricated and installed to all requirements for slip-critical connections.	B	$120 \times 10^8$	16 (110)	In net section originating at side of hole
2.3 Base metal at the net section of other mechanically fastened joints except eye bars and pin plates.	D	$22 \times 10^8$	7 (48)	In net section originating at side of hole
2.4 Base metal at net section of eyebar head or pin plate.	E	$11 \times 10^8$	4.5 (31)	In net section originating at side of hole

TABLE A-3.1 (cont'd)  
Fatigue Design Parameters

TABLE A-3.1

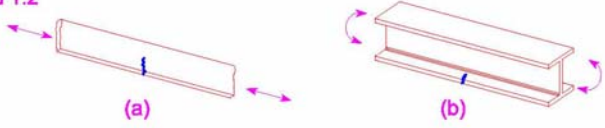
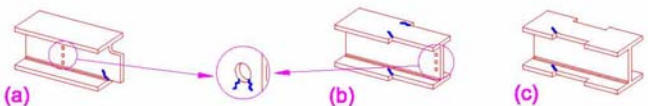
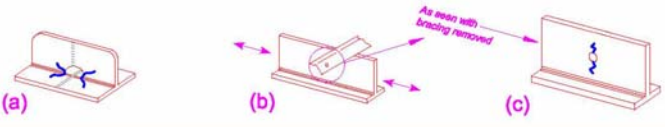
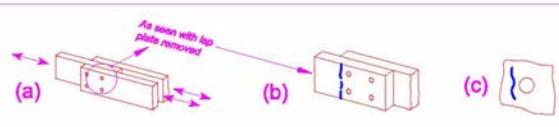
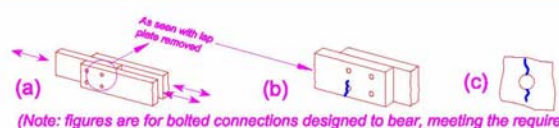
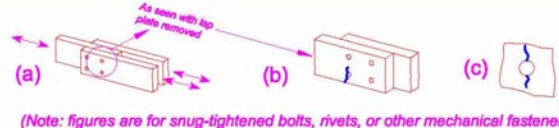

1.1 and 1.2	
1.3	
1.4	
2.1	 (Note: figures are for slip critical bolted connections)
2.2	 (Note: figures are for bolted connections designed to bear, meeting the requirements of slip critical connections)
2.3	 (Note: figures are for snug-tightened bolts, rivets, or other mechanical fasteners)
2.4	

TABLE A-3.1 (Cont'd)

## Fatigue Design Parameters

Description	Stress Category	Constant $C_f$	Threshold $F_{TH}$ ksi (MPa)	Potential Crack Initiation Point
SECTION 3 – WELDED JOINTS JOINING COMPONENTS OF BUILT-UP MEMBERS				
3.1 Base metal and weld metal in members without attachments built up of plates or shapes connected by continuous longitudinal complete-joint - penetration groove welds, back gouged and welded from second side, or by continuous fillet welds.	B	$120 \times 10^8$	16 (110)	From surface or internal discontinuities in weld away from end of weld
3.2 Base metal and weld metal in members without attachments built up of plates or shapes, connected by continuous longitudinal complete-joint-penetration groove welds with backing bars not removed, or by continuous partial-joint -penetration groove welds.	B'	$61 \times 10^8$	12 (83)	From surface or internal discontinuities in weld, including weld attaching backing bars
3.3 Base metal at weld metal terminations of longitudinal welds at weld access holes in connected built-up members.	D	$22 \times 10^8$	7 (48)	From the weld termination into the web or flange
3.4 Base metal at ends of longitudinal intermittent fillet weld segments.	E	$11 \times 10^8$	4.5 (31)	In connected material at start and stop locations of any weld deposit
3.5 Base metal at ends of partial length welded coverplates narrower than the flange having square or tapered ends, with or without welds across the ends; and coverplates wider than the flange with welds across the ends.  Flange thickness $\leq 0.8$ in. (20 mm)  Flange thickness $> 0.8$ in. (20 mm)	E  E'	$11 \times 10^8$  $3.9 \times 10^8$	4.5 (31) 2.6 (18)	In flange at toe of end weld or in flange at termination of longitudinal weld or in edge of flange with wide coverplates
3.6 Base metal at ends of partial length welded coverplates wider than the flange without welds across the ends.	E'	$3.9 \times 10^8$	2.6 (18)	In edge of flange at end of coverplate weld
SECTION 4 – LONGITUDINAL FILLET WELDED END CONNECTIONS				
4.1 Base metal at junction of axially loaded members with longitudinally welded end connections. Welds shall be on each side of the axis of the member to balance weld stresses. $t \leq 0.8$ in. (20 mm)	E	$11 \times 10^8$	4.5 (31)	Initiating from end of any weld termination extending into the base metal
$t > 0.8$ in. (20 mm)	E'	$3.9 \times 10^8$	2.6 (18)	

TABLE A-3.1 (Cont'd)  
Fatigue Design Parameters

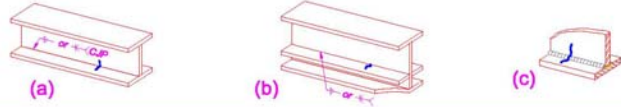
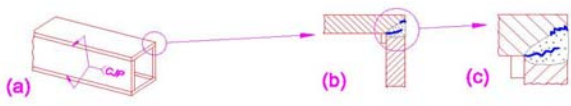


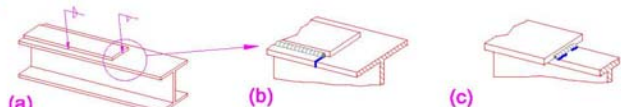

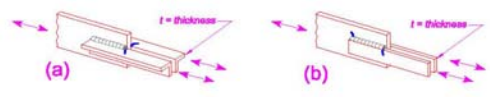
TABLE A-3.1	<p>3.1</p>  <p>(a) (b) (c)</p>
	<p>3.2</p>  <p>(a) (b) (c)</p>
	<p>3.3</p>  <p>(a) (b)</p>
	<p>3.4</p>  <p>(a) (b)</p>
	<p>3.5</p>  <p>(a) (b) (c)</p>
	<p>3.6</p>  <p>(a) (b)</p>
	<p>4.1</p>  <p>(a) (b)</p>

TABLE A-3.1 (Cont'd)

## Fatigue Design Parameters

Description	Stress Category	Constant $C_f$	Threshold $F_{TH}$ ksi (MPa)	Potential Crack Initiation Point
SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS				
5.1 Weld metal and base metal in or adjacent to complete-joint-penetration groove welded splices in rolled or welded cross sections with welds ground essentially parallel to the direction of stress and with soundness established by radiographic or ultrasonic inspection in accordance with the requirements of Sections 6.12 or 6.13 of AWS D1.1.	B	$120 \times 10^8$	16 (110)	From internal discontinuities in weld metal or along the fusion boundary
5.2 Weld metal and base 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 1:2½ and with weld soundness established by radiographic or ultrasonic inspection in accordance with the requirements of Sections 6.12 or 6.13 of AWS D1.1. $F_y < 90$ ksi (620 MPa)  $F_y \geq 90$ ksi (620 MPa)	B  B'	$120 \times 10^8$  $61 \times 10^8$	16 (110)  12 (83)	From internal discontinuities in filler metal or along fusion boundary or at start of transition when $F_y \geq 90$ ksi (620 MPa)
5.3 Base metal with $F_y$ equal to or greater than 90 ksi (620 MPa) and weld metal in or adjacent to complete-joint-penetration groove welded splices with welds ground essentially parallel to the direction of stress at transitions in width made on a radius of not less than 2 ft (600 mm) with the point of tangency at the end of the groove weld and with weld soundness established by radiographic or ultrasonic inspection in accordance with the requirements of Sections 6.12 or 6.13 of AWS D1.1.	B	$120 \times 10^8$	16 (110)	From internal discontinuities in filler metal or discontinuities along the fusion boundary
5.4 Weld metal and base metal in or adjacent to the toe of complete-joint-penetration groove welds in T or corner joints or splices, with or without transitions in thickness having slopes no greater than 1:2½, when weld reinforcement is not removed and with weld soundness established by radiographic or ultrasonic inspection in accordance with the requirements of Sections 6.12 or 6.13 of AWS D1.1.	C	$44 \times 10^8$	10 (69)	From surface discontinuity at toe of weld extending into base metal or into weld metal.



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TABLE A-3.1 (Cont'd)  
Fatigue Design Parameters

TABLE A-3.1

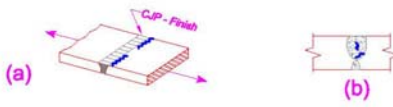
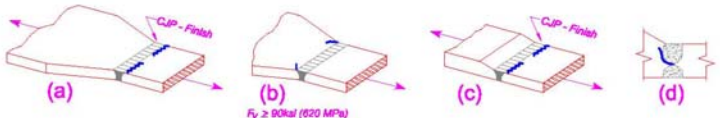

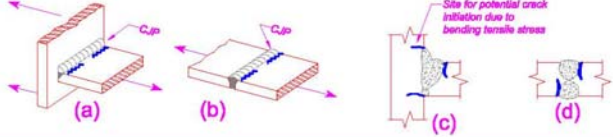
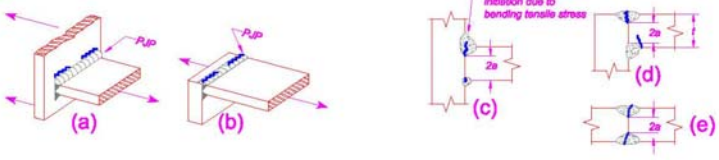
5.1	
5.2	 <p><math>F_y \geq 90 \text{ ksi (620 MPa)}</math> Cat. B'</p>
5.3	 <p><math>R \geq 2'-0"</math> (600 mm) <math>F_y \geq 90 \text{ ksi (620 MPa)}</math> Cat. B'</p>
5.4	 <p>Site for potential crack initiation due to bending tensile stress</p>
5.5	 <p>Site for potential crack initiation due to bending tensile stress</p>

TABLE A-3.1 (Cont'd)  
Fatigue Design Parameters

Description	Stress Category	Constant $C_f$	Threshold $F_{TH}$ ksi (MPa)	Potential Crack Initiation Point
SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS (cont'd)				
5.6 Base metal and weld metal at transverse end connections of tension-loaded plate elements using a pair of fillet welds on opposite sides of the plate. $F_{SR}$ shall be the smaller of the toe crack or root crack stress range. Crack initiating from weld toe:	C	$44 \times 10^8$	10 (69)	Initiating from geometrical discontinuity at toe of weld extending into base metal.
Crack initiating from weld root:	C"	Eqn. A-3-5 or A-3-5M	None provided	Initiating at weld root subject to tension extending into and through weld
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	$44 \times 10^8$	10 (69)	From geometrical discontinuity at toe of fillet extending into base metal
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS				

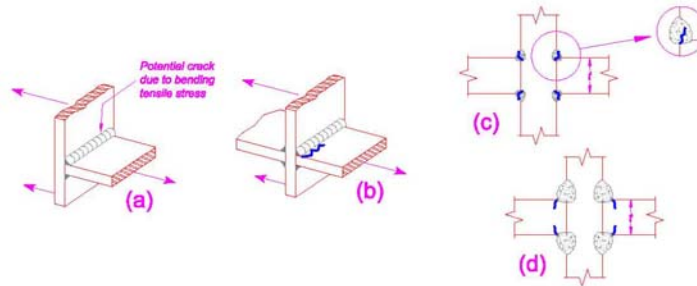
<p>6.1 Base metal at details attached by complete-joint-penetration groove welds subject to longitudinal loading only when the detail embodies a transition radius <math>R</math> with the weld termination ground smooth and with weld soundness established by radiographic or ultrasonic inspection in accordance with the requirements of 6.12 or 6.13 of AWS D1.1.</p> <p><math>R \geq 24</math> in. (600 mm)</p> <p>24 in. <math>&gt; R \geq 6</math> in. (600 mm <math>&gt; R \geq 150</math> mm)</p> <p>6 in. <math>&gt; R \geq 2</math> in. (150 mm <math>&gt; R \geq 50</math> mm)</p> <p>2 in. (50 mm) <math>&gt; R</math></p>	<p>B</p> <p>C</p> <p>D</p> <p>E</p>	<p><math>120 \times 10^8</math></p> <p><math>44 \times 10^8</math></p> <p><math>22 \times 10^8</math></p> <p><math>11 \times 10^8</math></p>	<p>16 (110)</p> <p>10 (69)</p> <p>7 (48)</p> <p>4.5 (31)</p>	<p>Near point of tangency of radius at edge of member</p>
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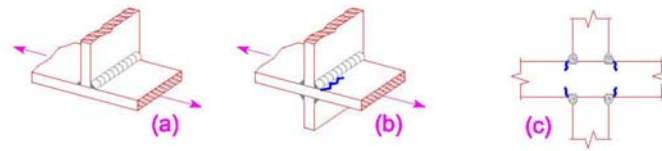
TABLE A-3.1 (Cont'd)  
Fatigue Design Parameters

TABLE A-3.1

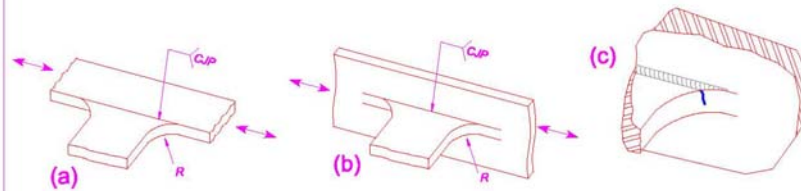
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**TABLE A-3.1 (Cont'd)**  
**Fatigue Design Parameters**

Description	Stress Category	Constant $C_f$	Threshold $F_{TH}$ ksi (MPa)	Potential Crack Initiation Point
<b>SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)</b>				
<p>6.2 Base metal at details of equal thickness attached by complete-joint-penetration groove welds subject to transverse loading with or without longitudinal loading when the detail embodies a transition radius <math>R</math> with the weld termination ground smooth and with weld soundness established by radiographic or ultrasonic inspection in accordance with the requirements of Sections 6.12 or 6.13 of AWS D1.1:</p> <p>When weld reinforcement is removed:</p> <p><math>R \geq 24</math> in. (600 mm)  <math>24</math> in. <math>&gt; R \geq 6</math> in.  (600 mm <math>&gt; R \geq 150</math> mm)</p> <p><math>6</math> in. <math>&gt; R \geq 2</math> in.  (150 mm <math>&gt; R \geq 50</math> mm)</p> <p><math>2</math> in. (50 mm) <math>&gt; R</math></p> <p>When weld reinforcement is not removed:</p> <p><math>R \geq 24</math> in. (600 mm)</p> <p><math>24</math> in. <math>&gt; R \geq 6</math> in.  (600 mm <math>&gt; R \geq 150</math> mm)</p> <p><math>6</math> in. <math>&gt; R \geq 2</math> in.  (150 mm <math>&gt; R \geq 50</math> mm)</p> <p><math>2</math> in. (50 mm) <math>&gt; R</math></p>	<p>B</p> <p>C</p> <p>D</p> <p>E</p> <p>C</p> <p>C</p> <p>D</p> <p>E</p>	<p><math>120 \times 10^8</math></p> <p><math>44 \times 10^8</math></p> <p><math>22 \times 10^8</math></p> <p><math>11 \times 10^8</math></p> <p><math>44 \times 10^8</math></p> <p><math>44 \times 10^8</math></p> <p><math>22 \times 10^8</math></p> <p><math>11 \times 10^8</math></p>	<p>16 (110)</p> <p>10 (69)</p> <p>7 (48)</p> <p>4.5 (31)</p> <p>10 (69)</p> <p>10 (69)</p> <p>7 (48)</p> <p>4.5 (31)</p>	<p>Near points of tangency of radius or in the weld or at fusion boundary or member or attachment</p> <p>At toe of the weld either along edge of member or the attachment</p>
<p>6.3 Base metal at details of unequal thickness attached by complete-joint-penetration groove welds subject to transverse loading with or without longitudinal loading when the detail embodies a transition radius <math>R</math> with the weld termination ground smooth and with weld soundness established by radiographic or ultrasonic inspection in accordance with the requirements of Sections 6.12 or 6.13 of AWS D1.1.</p> <p>When weld reinforcement is removed:</p> <p><math>R &gt; 2</math> in. (50 mm)</p> <p><math>R \leq 2</math> in. (50 mm)</p> <p>When reinforcement is not removed:</p> <p>Any radius</p>	<p>D</p> <p>E</p> <p>E</p>	<p><math>22 \times 10^8</math></p> <p><math>11 \times 10^8</math></p> <p><math>11 \times 10^8</math></p>	<p>7 (48)</p> <p>4.5 (31)</p> <p>4.5 (31)</p>	<p>At toe of weld along edge of thinner material</p> <p>In weld termination in small radius</p> <p>At toe of weld along edge of</p>

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				thinner material
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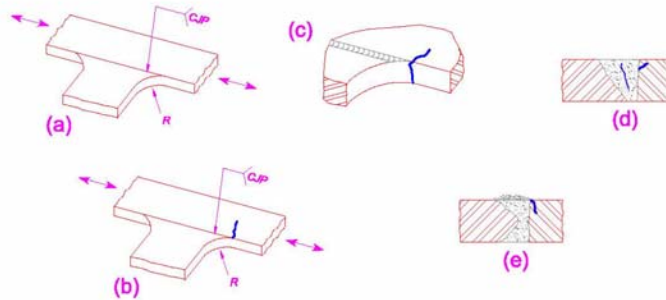
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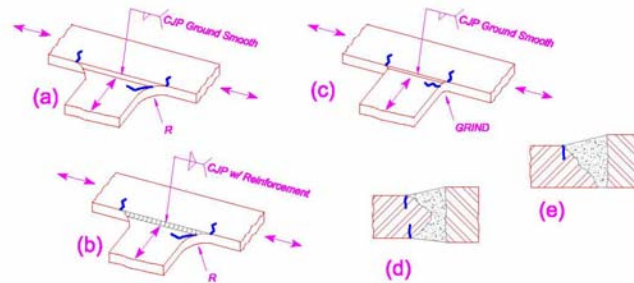
TABLE A-3.1 (Cont'd)  
Fatigue Design Parameters



TABLE A-3.1 6.2



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TABLE A-3.1 (Cont'd)  
Fatigue Design Parameters

Description	Stress Category	Constant $C_f$	Threshold $F_{TH}$ ksi (MPa)	Potential Crack Initiation Point
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)				
6.4 Base metal subject to longitudinal stress at transverse members, with or without transverse stress, attached by fillet or partial penetration groove welds parallel to direction of stress when the detail embodies a transition radius, $R$ , with weld termination ground smooth:				Initiating in base metal at the weld termination or at the toe of the weld extending into the base metal
$R > 2$ in. (50 mm)	D	$22 \times 10^8$	7 (48)	
$R \leq 2$ in. (50 mm)	E	$11 \times 10^8$	4.5 (31)	
SECTION 7 – BASE METAL AT SHORT ATTACHMENTS <sup>1</sup>				
7.1 Base metal subject to longitudinal loading at details <u>with</u> 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 thickness of the attachment, $b$ :				Initiating in base metal at the weld termination or at the toe of the weld extending into the base metal
$a < 2$ in. (50 mm)	C	$44 \times 10^8$	10 (69)	
$2$ in. (50 mm) $\leq a \leq$ lesser of $12b$ or $4$ in. (100 mm)	D	$22 \times 10^8$	7 (48)	
$a > 4$ in. (100 mm) when $b > 0.8$ in. (20 mm)	E	$11 \times 10^8$	4.5 (31)	
$a >$ lesser of $12b$ or $4$ in. (100 mm) when $b \leq 0.8$ in. (20 mm)	E'	$3.9 \times 10^8$	2.6 (18)	
7.2 Base metal subject to longitudinal stress at details attached by fillet or partial-joint-penetration groove welds, with or without transverse load on detail, when the detail embodies a transition radius, $R$ , with weld termination ground smooth:				Initiating in base metal at the weld termination, extending into the base metal
$R > 2$ in. (50 mm)	D	$22 \times 10^8$	7 (48)	
$R \leq 2$ in. (50 mm)	E	$11 \times 10^8$	4.5 (31)	
<sup>1</sup> "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.				

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TABLE A-3.1 (Cont'd)  
Fatigue Design Parameters

TABLE A-3.1

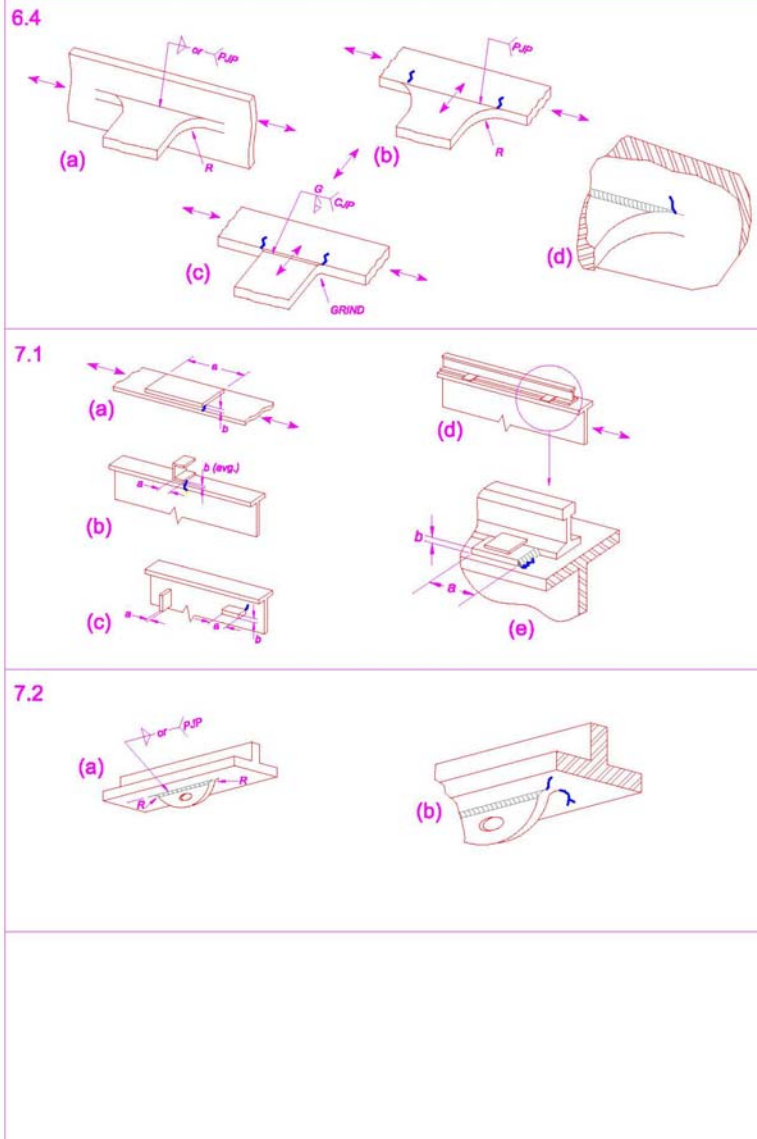



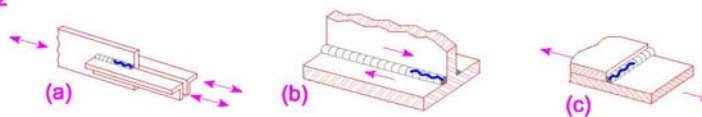

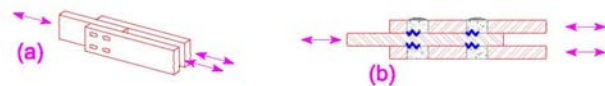
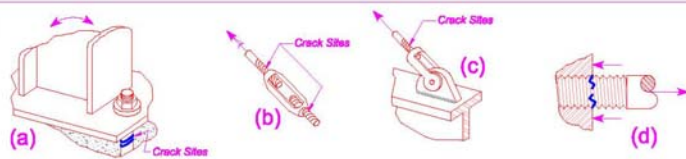
TABLE A-3.1 (Cont'd)

## Fatigue Design Parameters

Description	Stress Category	Constant $C_f$	Threshold $F_{TH}$ ksi (MPa)	Potential Crack Initiation Point
SECTION 8 - MISCELLANEOUS				
8.1 Base metal at stud-type shear connectors attached by fillet or automatic stud welding.	C	$44 \times 10^8$	10 (69)	At toe of weld in base metal
8.2 Shear on throat of continuous or intermittent longitudinal or transverse fillet welds.	F	$150 \times 10^{10}$ (Eqn. A-3-2 or A-3-2M)	8 (55)	Initiating at the root of the fillet weld, extending into the weld
8.3 Base metal at plug or slot welds.	E	$11 \times 10^8$	4.5 (31)	Initiating in the base metal at the end of the plug or slot weld, extending into the base metal
8.4 Shear on plug or slot welds.	F	$150 \times 10^{10}$ (Eqn. A-3-2 or A-3-2M)	8 (55)	Initiating in the weld at the <i>faying surface</i> , extending into the weld
8.5 Snug-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.	G	$3.9 \times 10^8$	7 (48)	Initiating at the root of the threads, extending into the fastener

TABLE A-3.1 (Cont'd)  
Fatigue Design Parameters

TABLE A-3.1

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8.4	
8.5	

## APPENDIX 4

### STRUCTURAL DESIGN FOR FIRE CONDITIONS

This appendix provides criteria for the design and evaluation of *structural steel* components, systems and frames for fire conditions. These criteria provide for the determination of the heat input, thermal expansion and degradation in mechanical properties of materials at elevated temperatures that cause progressive decrease in strength and *stiffness* of structural components and systems at elevated temperatures.

The appendix is organized as follows:

- 4.1. General Provisions
- 4.2 Structural Design for Fire Conditions by Analysis
- 4.3. Design by Qualification Testing

#### 4.1. GENERAL PROVISIONS

The methods contained in this appendix provide regulatory evidence of compliance in accordance with the design applications outlined in this section.

The appendix uses the following terms in addition to the terms in the Glossary.

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

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

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

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

*Fire.* Destructive burning, as manifested by any or all of the following: light, flame, heat, or smoke.

*Fire barrier.* Element of construction formed of fire-resisting materials and tested in accordance with an approved standard fire resistance test, to demonstrate compliance with the *applicable building code*.

*Fire endurance.* Measure of the elapsed time during which a material or assembly continues to exhibit *fire resistance*.

*Fire resistance.* 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.

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

*Heat flux.* Radiant energy per unit surface area.

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

*Performance-based design.* 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.

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

*Unrestrained construction.* Floor and roof assemblies and individual *beams* in buildings that are assumed to be free to rotate and expand throughout the range of anticipated *elevated temperatures*.

## 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*.

## 2. Design by Engineering Analysis

The analysis methods in Section 4.2 are permitted to be used to document



the anticipated performance of steel framing when subjected to design-basis fire scenarios. Methods in Section 4.2 provide evidence of compliance with performance objectives established in Section 4.1.1.

The analysis methods in Section 4.2 are permitted to be used to demonstrate an equivalency for an alternative material or method, as permitted by the *applicable building code*.

Structural design for fire conditions using Appendix 4.2 shall be performed using the load and resistance factor design method in accordance with the provisions of Section B3.3 (LRFD).

### 3. Design by Qualification Testing

The qualification testing methods in Section 4.3 are permitted to be used to document the fire resistance of steel framing subject to the standardized fire testing protocols required by the *applicable building code*.

### 4. Load Combinations and Required Strength

The *required strength* of the structure and its elements shall be determined from the *gravity load combination* as follows:

$$[0.9 \text{ or } 1.2] D + T + 0.5 L + 0.2 S \quad (\text{A-4-1})$$

where

$D$  = nominal dead load

$L$  = nominal occupancy live load

$S$  = nominal snow load

$T$  = nominal forces and deformations due to the design-basis fire defined in Section 4.2.1

A *notional load*,  $N_i = 0.002Y_i$ , as defined in Section C2.2, where  $N_i$  = notional load applied at framing level  $i$  and  $Y_i$  = *gravity load* from combination A-4-1 acting on framing level  $i$ , shall be applied in combination with the loads stipulated in Equation A-4-1. Unless otherwise stipulated by the applicable building code,  $D$ ,  $L$  and  $S$  shall be the *nominal loads* specified in SEI/ASCE 7.

## 4.2. STRUCTURAL DESIGN FOR FIRE CONDITIONS BY ANALYSIS

It is permitted to design structural members, components and building frames for *elevated temperatures* in accordance with the requirements of this section.

### 1. Design-Basis Fire

A *design-basis fire* shall be identified to describe the heating conditions for the structure. These heating conditions shall relate to the fuel commodities

and compartment characteristics present in the assumed fire area. The fuel *load* density based on the occupancy of the space shall be considered when determining the total fuel load. Heating conditions shall be specified either in terms of a *heat flux* or temperature of the upper gas layer created by the *fire*. The variation of the heating conditions with time shall be determined for the duration of the fire.

When the analysis methods in Section 4.2 are used to demonstrate an equivalency as an alternative material or method as permitted by the *applicable building code*, the design-basis fire shall be determined in accordance with ASTM E119.

#### 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.

#### 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 of the space (natural and mechanical), compartment dimensions and thermal characteristics of the compartment boundary.

The fire duration in a particular area shall be determined by considering the total combustible mass, or fuel *load* available in the space. In the case of either a localized fire or a post-flashover compartment fire, the fire duration shall be determined as the total combustible mass divided by the mass loss rate.

#### 1.3. Exterior Fires

The exposure of exterior structure to flames projecting from windows or other wall openings as a result of a post-flashover compartment fire shall be considered along with the radiation from the interior fire through the opening. The shape and length of the flame projection shall be used along with the distance between the flame and the exterior steelwork to determine the heat flux to the steel. The method identified in Section 4.2.1.2 shall be used for describing the characteristics of the interior compartment fire.

#### 1.4. 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.

## 2. Temperatures in Structural Systems under Fire Conditions

Temperatures within structural members, components and frames due to the heating conditions posed by the design-basis fire shall be determined by a heat transfer analysis.

## 3. Material Strengths at Elevated Temperatures

Material properties at elevated temperatures shall be determined from test data. In the absence of such data, it is permitted to use the material properties stipulated in this section. These relationships do not apply for steels with a *yield strength* in excess of 65 ksi (448 MPa) or concretes with specified compression strength in excess of 8,000 psi (55 MPa).

### 3.1. Thermal Elongation

The coefficients of expansion shall be taken as follows:

(a) For structural and reinforcing steels: For calculations at temperatures above 150 °F (65 °C), the coefficient of thermal expansion shall be  $7.8 \times 10^{-6}/^{\circ}\text{F}$  ( $1.4 \times 10^{-5}/^{\circ}\text{C}$ ).

(b) For normal weight concrete: For calculations at temperatures above 150 °F (65 °C), the coefficient of thermal expansion shall be  $1.0 \times 10^{-5}/^{\circ}\text{F}$  ( $1.8 \times 10^{-5}/^{\circ}\text{C}$ ).

(c) For lightweight concrete: For calculations at temperatures above 150 °F (65 °C), the coefficient of thermal expansion shall be  $4.4 \times 10^{-6}/^{\circ}\text{F}$  ( $7.9 \times 10^{-6}/^{\circ}\text{C}$ ).

### 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  $F_y(T)$ ,  $F_p(T)$ ,  $F_u(T)$ ,  $E(T)$ ,  $G(T)$ ,  $f'_c(T)$ ,  $E_c(T)$  and  $\epsilon_{cu}(T)$  at elevated temperature to be used in *structural analysis*, expressed as the ratio with respect to the property at ambient, assumed to be 68 °F (20 °C), shall be defined as in Tables A-4.2.1 and A-4.2.2.  $F_p(T)$  is the proportional limit at elevated temperatures, which is calculated as a ratio to yield strength as specified in Table A-4.2.1. It is permitted to interpolate between these values.

For lightweight concrete (LWC), values of  $\varepsilon_{cu}$  shall be obtained from tests.

Table A-4.2.1

Properties of Steel at Elevated Temperatures

Steel Temperature (°F) [°C]	$k_E = E(T) / E$ $= G(T) / G$	$k_p = F_p(T) / F_y$	$k_y = F_y(T) / F_y$	$k_u = F_u(T) / F_y$
68 [20]	*	*	*	*
200 [93]	1.00	1.00	*	*
400 [204]	0.90	0.81	*	*
600 [316]	0.78	0.61	*	*
750 [399]	0.70	0.42	1.00	1.00
800 [427]	0.67	0.36	0.94	0.94
1000 [538]	0.49	0.18	0.66	0.66
1200 [649]	0.22	0.08	0.35	0.35
1400 [760]	0.11	0.05	0.16	0.16
1600 [871]	0.07	0.04	0.07	0.07
1800 [982]	0.05	0.03	0.04	0.04
2000 [1093]	0.02	0.01	0.02	0.02
2200 [1204]	0.00	0.00	0.00	0.00
*Use ambient properties.				

Table A-4.2.2

Properties of Concrete at Elevated Temperatures

Concrete Temperature (°F) [°C]	$k_c = f'_c(T) / f'_c$		$E_c(T) / E_c$	$\varepsilon_{cu}(T) (\%)$
	NWC	LWC		NWC
68 [20]	1.00	1.00	1.00	0.25
200 [93]	0.95	1.00	0.93	0.34
400 [204]	0.90	1.00	0.75	0.46
550 [288]	0.86	1.00	0.61	0.58
600 [316]	0.83	0.98	0.57	0.62
800 [427]	0.71	0.85	0.38	0.80
1000 [538]	0.54	0.71	0.20	1.06
1200 [649]	0.38	0.58	0.092	1.32
1400 [760]	0.21	0.45	0.073	1.43
1600 [871]	0.10	0.31	0.055	1.49
1800 [982]	0.05	0.18	0.036	1.50
2000 [1093]	0.01	0.05	0.018	1.50
2200 [1204]	0.00	0.00	0.00	-

## 4. Structural Design Requirements

### 4.1. General Structural Integrity

The structural frame shall be capable of providing adequate strength and deformation capacity to withstand, as a system, the structural actions developed during the fire within the prescribed limits of deformation. The *structural system* shall be designed to sustain local damage with the structural system as a whole remaining stable.

Continuous *load* paths shall be provided to transfer all *forces* from the exposed region to the final point of resistance. The foundation shall be designed to resist the forces and to accommodate the deformations developed during the design-basis fire.

### 4.2. Strength Requirements and Deformation Limits

Conformance of the structural system to these requirements shall be demonstrated by constructing a mathematical model of the structure based on principles of structural mechanics and evaluating this model for the internal forces and deformations in the members of the structure developed by the temperatures from the design-basis fire.

Individual members shall be provided with adequate strength to resist the shears, axial forces, and moments determined in accordance with these provisions.

*Connections* shall develop the strength of the connected members or the forces indicated above. Where the means of providing *fire resistance* requires the consideration of deformation criteria, the deformation of the structural system, or members thereof, under the design-basis fire shall not exceed the prescribed limits.

### 4.3. Methods of Analysis

#### 4.3a. Advanced Methods of Analysis

The methods of analysis in this section are permitted for the design of all steel building structures for fire conditions. The design-basis fire exposure shall be that determined in Section 4.2.1. The analysis shall include both a thermal response and the mechanical response to the design-basis fire.

The *thermal response* shall produce a temperature field in each structural element as a result of the design-basis fire and shall incorporate temperature-dependent thermal properties of the structural elements and fire-resistive materials, as per Section 4.2.2.

The *mechanical response* results in forces and deformations in the

structural system subjected to the thermal response calculated from the design-basis fire. The mechanical response shall take into account explicitly the deterioration in strength and *stiffness* with increasing temperature, the effects of thermal expansions and large deformations. Boundary conditions and connection fixity must represent the proposed structural design. Material properties shall be defined as per Section 4.2.3.

The resulting analysis shall consider all relevant *limit states*, such as excessive deflections, connection fractures and overall or *local buckling*.

#### 4.3b. Simple Methods of Analysis

The methods of analysis in this section are permitted to be used for the evaluation of the performance of individual members at elevated temperatures during exposure to fire.

The support and restraint conditions (forces, moments and boundary conditions) applicable at normal temperatures are permitted to be assumed to remain unchanged throughout the fire exposure.

##### (1) Tension Members

It is permitted to model the thermal response of a tension element using a one-dimensional heat transfer equation with heat input as directed by the design-basis fire defined in Section 4.2.1.

The *design strength* of a tension member shall be determined using the provisions of Chapter D, with steel properties as stipulated in Section 4.2.3 and assuming a uniform temperature over the cross section using the temperature equal to the maximum steel temperature.

##### (2) Compression Members

It is permitted to model the thermal response of a compression element using a one-dimensional heat transfer equation with heat input as directed by the design-basis fire defined in Section 4.2.1.

The design strength of a compression member shall be determined using the provisions of Chapter E with steel properties as stipulated in Section 4.2.3, with the following exception. For steel temperatures equal to or greater than 400°F (204°C), Equation A-4-2 shall be used in lieu of Equations E3-2 and E3-3 to calculate the nominal compressive strength for flexural buckling:

$$F_{cr}(T) = \left[ 0.42 \sqrt{\frac{F_y(T)}{F_e(T)}} \right] F_y(T) \quad (\text{A-4-2})$$

where  $F_y(T)$  is the yield stress at elevated temperature and  $F_e(T)$  is

the critical elastic buckling stress calculated from Equation E3-4 with the elastic modulus  $E(T)$  at elevated temperature.  $F_y(T)$  and  $E(T)$  are obtained using coefficients from Table A-4.2.1.

### (3) Flexural Members

It is permitted to model the thermal response of flexural elements using a one-dimensional heat transfer equation to calculate bottom flange temperature and to assume that this bottom flange temperature is constant over the depth of the member.

The design strength of a flexural member shall be determined using the provisions of Chapter F with steel properties as stipulated in Section 4.2.3, with the following exception. For steel temperatures equal to or greater than 400 °F (204 °C), Equations A-4-3 through A-4-10alt shall be used in lieu of Equations F2-2 through F2-6 to calculate the nominal flexural strength for lateral-torsional buckling of laterally unbraced doubly symmetric members:

(a) when  $L_b \leq L_r$

$$M_n(T) = C_b \left[ M_r(T) + [M_p(T) - M_r(T)] \left( 1 - \frac{L_b}{L_r(T)} \right)^{c_x(T)} \right] \quad (\text{A-4-3})$$

(b) when  $L_b > L_r(T)$

$$M_n(T) = F_{cr}(T) S_x \quad (\text{A-4-4})$$

where

$$F_{cr}(T) = \frac{C_b \pi^2 E(T)}{\left( \frac{L_b}{r_{ts}} \right)^2} \sqrt{1 + 0.078 \frac{Jc}{S_x h_o} \left( \frac{L_b}{r_{ts}} \right)^2} \quad (\text{A-4-5})$$

$$L_r(T) \geq 1.95 r_{ts} \frac{E(T)}{F_L(T)} \sqrt{\frac{Jc}{S_x h_o} \sqrt{\left| \frac{Jc}{S_x h_o} \right|^2} \sqrt{6.76 \left| \frac{F_L(T)}{E(T)} \right|^2}} \quad (\text{A-4-6})$$

$$M_r(T) = S_x F_L(T) \quad (\text{A-4-7})$$

$$F_L(T) = F_y (k_p(T) - 0.3 k_y(T)) \quad (\text{A-4-8})$$

$$M_p(T) = Z_x F_y(T) \quad (\text{A-4-9})$$

$$c_x(T) = 0.53 + \frac{T}{450} \leq 3.0 \quad \text{where } T \text{ is in } ^\circ\text{F} \quad (\text{A-4-10})$$

$$c_x(T) = 0.6 + \frac{T}{250} \leq 3.0 \quad \text{where } T \text{ is in } ^\circ\text{C} \quad (\text{A-4-10M})$$

The material properties at elevated temperatures [ $E(T)$  and  $F_y(T)$ ] and the  $k_p(T)$  and  $k_y(T)$  coefficients are calculated in accordance with Table A-4.2.1, and other terms are as defined in Chapter F.

#### (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% from the mid-depth of the web to the top flange of the *beam*.

The design strength of a *composite* flexural member shall be determined using the provisions of Chapter I, with reduced *yield stresses* in the steel consistent with the temperature variation described under thermal response.

#### 4.4. Design Strength

The design strength shall be determined as in Section B3.3. The *nominal strength*,  $R_n$ , shall be calculated using material properties, as stipulated in Section 4.2.3, at the temperature developed by the design-basis fire.

#### 4.3. DESIGN BY QUALIFICATION TESTING

##### 1. Qualification Standards

Structural members and components in steel buildings shall be qualified for the rating period in conformance with ASTM E119. Demonstration of compliance with these requirements using the procedures specified for steel construction in Section 5 of SEI/ASCE/ SFPE Standard 29-05, *Standard Calculation Methods for Structural Fire Protection*, is permitted.

##### 2. Restrained Construction

For floor and roof assemblies and individual *beams* in buildings, a restrained condition exists when the surrounding or supporting structure is capable of resisting forces and accommodating deformations caused by thermal expansion throughout the range of anticipated *elevated temperatures*.



Steel beams, girders and frames supporting concrete slabs that are welded or bolted to integral framing members shall be considered *restrained construction*.

**3. Unrestrained Construction**

Steel beams, girders and frames that do not support a concrete slab shall be considered unrestrained unless the members are bolted or welded to surrounding construction that has been specifically designed and detailed to resist effects of elevated temperatures.

A steel member bearing on a wall in a single span or at the end span of multiple spans shall be considered unrestrained unless the wall has been designed and detailed to resist effects of thermal expansion.

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## APPENDIX 5 EVALUATION OF EXISTING STRUCTURES

This appendix applies to the evaluation of the strength and *stiffness* under static vertical (gravity) *loads* of existing structures by *structural analysis*, by load tests, or by a combination of structural analysis and load tests when specified by the engineer of record or in the contract documents. For such evaluation, the steel grades are not limited to those listed in Section A3.1. This appendix does not address load testing for the effects of seismic loads or moving loads (vibrations).

The Appendix is organized as follows:

- 5.1. General Provisions
- 5.2. Material Properties
- 5.3. Evaluation by Structural Analysis
- 5.4. Evaluation by Load Tests
- 5.5. Evaluation Report

### 5.1. GENERAL PROVISIONS

These provisions shall be applicable when the evaluation of an existing steel structure is specified for (a) verification of a specific set of design loadings or (b) determination of the *available strength* of a *load* resisting member or system. The evaluation shall be performed by *structural analysis* (Section 5.3), by load tests (Section 5.4), or by a combination of structural analysis and load tests, as specified in the contract documents. Where load tests are used, the *engineer of record* shall first analyze the applicable parts of the structure, prepare a testing plan, and develop a written procedure to prevent excessive permanent deformation or catastrophic collapse during testing.

### 5.2. MATERIAL PROPERTIES

#### 1. Determination of Required Tests

The *engineer of record* shall determine the specific tests that are required from Section 5.2.2 through 5.2.6 and specify the locations where they are required. Where available, the use of applicable project records shall be permitted to reduce or eliminate the need for testing.

#### 2. Tensile Properties

Tensile properties of members shall be considered in evaluation by *structural analysis* (Section 5.3) or *load* tests (Section 5.4). Such properties shall include the *yield stress*, *tensile strength* and *percent elongation*. Where available, certified material test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM A6/A6M or A568/A568M, as applicable, shall be permitted for this purpose. Otherwise, tensile tests shall be conducted in accordance with ASTM A370 from samples cut from components of the structure.

### 3. Chemical Composition

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

### 4. Base Metal Notch Toughness

Where welded tension splices in heavy shapes and plates as defined in Section A3.1d are critical to the performance of the structure, the *Charpy V-Notch toughness* shall be determined in accordance with the provisions of Section A3.1d. If the notch toughness so determined does not meet the provisions of Section A3.1d, the *engineer of record* shall determine if remedial actions are required.

### 5. Weld Metal

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

### 6. Bolts and Rivets

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

## 5.3. EVALUATION BY STRUCTURAL ANALYSIS

### 1. Dimensional Data

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

### 2. Strength Evaluation

*Forces (load effects)* in members and connections shall be determined by *structural analysis* applicable to the type of structure evaluated. The load effects shall be determined for the *loads* and *factored load combinations*

stipulated in Section B2.

The *available strength* of members and connections shall be determined from applicable provisions of Chapters B through K of this Specification.

### 3. Serviceability Evaluation

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

## 5.4. EVALUATION BY LOAD TESTS

### 1. Determination of Load Rating by Testing

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

The tested strength of the structure shall be taken as the maximum applied test load plus the in-situ dead load. The live load rating of a floor structure shall be determined by setting the tested strength equal to  $1.2D + 1.6L$ , where  $D$  is the nominal dead load and  $L$  is the nominal live load rating for the structure. The nominal live load rating of the floor structure shall not exceed that which can be calculated using applicable provisions of the specification. For roof structures,  $L_r$ ,  $S$ , or  $R$  as defined in SEI/ASCE 7, shall be substituted for  $L$ . More severe *load combinations* shall be used where required by *applicable building codes*.

Periodic unloading shall be considered once the *service load* level is attained and after the onset of inelastic structural behavior is identified to document the amount of permanent set and the magnitude of the inelastic deformations. Deformations of the structure, such as member deflections, shall be monitored at critical locations during the test, referenced to the initial position before loading. It shall be demonstrated that the deformation of the structure does not increase by more than 10% during a one-hour holding period under sustained, maximum test load. It is permissible to repeat the sequence if necessary to demonstrate compliance.

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

### 2. Serviceability Evaluation

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

**5.5. EVALUATION REPORT**

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

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## APPENDIX 6

### STABILITY BRACING FOR COLUMNS AND BEAMS

This appendix addresses the minimum strength and *stiffness* necessary to provide a braced point in a column, beam or beam-column. It is organized as follows:

- 6.1. General Provisions
- 6.2. Column Bracing
- 6.3. Beam Bracing
- 6.4. Beam-Column Bracing

**User Note:** The stability requirements for braced-frame systems are provided in Chapter C. The provisions in this appendix apply to bracing that is provided to stabilize individual columns, beams and beam-columns.

#### 6.1. GENERAL PROVISIONS

Columns with end and intermediate braced points designed to meet the requirements in Section 6.2 are permitted to be designed based on the *unbraced length*,  $L$ , between the braced points with an *effective length factor*,  $K = 1.0$ . Beams with intermediate braced points designed to meet the requirements in Section 6.3 are permitted to be designed based on the unbraced length,  $L_b$ , between the braced points.

When bracing is perpendicular to the members to be braced, the equations in Sections 6.2 and 6.3 shall be used directly. When bracing is oriented at an angle to the member to be braced, these equations 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.

**User Note:** In this appendix, relative and nodal bracing systems are addressed for columns and for beams with lateral bracing. For beams with torsional bracing, nodal and continuous bracing systems are addressed.

A *relative brace* controls the movement of the braced point with respect to adjacent braced points. A *nodal brace* controls the movement at the braced point without direct interaction with adjacent braced points. A continuous bracing system consists of bracing that is attached along the entire member length; however, nodal bracing systems with a regular spacing can also be modeled as a continuous system.

The *available strength* and *stiffness* of the bracing members and connections shall equal or exceed the *required strength* and *stiffness*, respectively, unless analysis indicates that smaller values are justified. A *second-order analysis* that includes the initial out-of-straightness of the member to obtain brace strength and stiffness requirements is permitted in lieu of the requirements of this appendix.

#### 6.2. COLUMN BRACING

It is permitted to brace an individual *column* at end and intermediate points along the length using either relative or nodal bracing.

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## 1. Relative Bracing

The *required strength* is

$$P_{rb} = 0.004P_r \quad (\text{A-6-1})$$

The *required stiffness* is

$$\beta_{br} = \frac{1}{\phi} \left( \frac{2P_r}{L_b} \right) (\text{LRFD}) \quad \beta_{br} = \Omega \left( \frac{2P_r}{L_b} \right) (\text{ASD}) \quad (\text{A-6-2})$$

where

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

$L_b$  = distance between braces, in. (mm)

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

$P_r$  = required strength in axial compression using *LRFD load combinations*, kips (N)

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

$P_r$  = required strength in axial compression using *ASD load combinations*, kips (N)

## 2. Nodal Bracing

The *required strength* is

$$P_{rb} = 0.01P_r \quad (\text{A-6-3})$$

The *required stiffness* is

$$\beta_{br} = \frac{1}{\phi} \left( \frac{8P_r}{L_b} \right) (\text{LRFD}) \quad \beta_{br} = \Omega \left( \frac{8P_r}{L_b} \right) (\text{ASD}) \quad (\text{A-6-4})$$

**User Note:** These equations correspond to the assumption that *nodal braces* are equally spaced along the column.

where

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

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

$P_r$  = required strength in axial compression using *LRFD load combinations*, kips (N)

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

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$P_r$  = required strength in axial compression using *ASD load combinations*, kips (N)

In Equation A-6-4,  $L_b$  need not be taken less than the maximum *unbraced length*,  $KL$ , permitted for the column based upon the axial required strength,  $P_r$ .

### 6.3. BEAM BRACING

*Beams* and trusses shall be restrained against rotation about their longitudinal axis at points of support. When a braced point is assumed in the design between points of support, *lateral bracing*, *torsional bracing* or a combination of the two shall be provided to prevent the relative displacement of the top and bottom flanges (i.e., to prevent twist). In members subject to *double curvature* bending, the inflection point shall not be considered a braced point unless bracing is provided at that location.

#### 1. Lateral Bracing

Lateral bracing shall be attached at or near the beam compression flange, except as follows:

- (1) At the free end of a cantilevered beam, lateral bracing shall be attached at or near the top (tension) flange.
- (2) For braced beams subject to double curvature bending, lateral bracing shall be attached to both flanges at the braced point nearest the inflection point.

#### 1a. Relative Bracing

The *required strength* is

$$P_{rb} = 0.008M_r C_d / h_o \quad (\text{A-6-5})$$

The required *stiffness* is

$$\beta_{br} = \frac{1}{\phi} \left( \frac{4M_r C_d}{L_b h_o} \right) (\text{LRFD}) \quad \beta_{br} = \Omega \left( \frac{4M_r C_d}{L_b h_o} \right) (\text{ASD}) \quad (\text{A-6-6})$$

where

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

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

$C_d$  = 1.0 except in the following case;

= 2.0 for the brace closest to the inflection point in a beam subject to double curvature bending

$L_b$  = laterally *unbraced length*, in. (mm)

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

$M_r$  = flexural *required strength* using *LRFD load combinations*, kip-in. (N-mm)



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

$M_r$  = flexural *required strength* using *ASD load combinations*, kip-in. (N-mm)

**1b. Nodal Bracing**

The *required strength* is

$$P_{rb} = 0.02M_r C_d / h_o \quad (\text{A-6-7})$$

The *required stiffness* is

$$\beta_{br} = \frac{1}{\phi} \left( \frac{10M_r C_d}{L_b h_o} \right) (\text{LRFD}) \quad \beta_{br} = \Omega \left( \frac{10M_r C_d}{L_b h_o} \right) (\text{ASD}) \quad (\text{A-6-8})$$

where

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

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

$M_r$  = flexural *required strength* using *LRFD load combinations*, kip-in. (N-mm)

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

$M_r$  = flexural *required strength* using *ASD load combinations*, kip-in. (N-mm)

In Equation A-6-8,  $L_b$  need not be taken less than the maximum unbraced length permitted for the beam based upon the flexural required strength,  $M_r$ .

**2. Torsional Bracing**

It is permitted to attach torsional bracing at any cross-sectional location, and it need not be attached near the compression flange.

**User Note:** Torsional bracing can be provided with a moment-connected beam, cross-frame, or other diaphragm element.

**2a. Nodal Bracing**

The *required strength* is

$$M_{rb} = \frac{0.024M_r L}{nC_b L_b} \quad (\text{A-6-9})$$

The *required stiffness* is

$$\beta_{Tb} = \frac{\beta_T}{\left(1 - \frac{\beta_T}{\beta_{sec}}\right)} \quad (\text{A-6-10})$$

where

$$\beta_T = \frac{1}{\phi} \left( \frac{2.4LM_r^2}{nEI_y C_b^2} \right) (\text{LRFD}) \quad \beta_T = \Omega \left( \frac{2.4LM_r^2}{nEI_y C_b^2} \right) (\text{ASD}) \quad (\text{A-6-11})$$

$$\beta_{sec} = \frac{3.3E}{h_o} \left( \frac{1.5h_o t_w^3}{12} + \frac{t_{st} b_s^3}{12} \right) \quad (\text{A-6-12})$$

where

$$\phi = 0.75 (\text{LRFD}) \quad \Omega = 3.00 (\text{ASD})$$

**User Note:**  $\Omega = 1.5^2 / \phi = 3.00$  in Equation A-6-11 because the moment term is squared.

- $L$  = span length, in. (mm)
- $n$  = number of *nodal braced* points within the span
- $E$  = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)
- $I_y$  = out-of-plane moment of inertia, in.<sup>4</sup> (mm<sup>4</sup>)
- $C_b$  = modification factor defined in Chapter F
- $t_w$  = *beam* web thickness, in. (mm)
- $t_{st}$  = thickness of web stiffener, in. (mm)
- $b_s$  = *stiffener* width for one-sided stiffeners, in. (mm)  
= twice the individual *stiffener* width for pairs of stiffeners, in. (mm)
- $\beta_T$  = brace *stiffness* excluding web distortion, kip-in./rad (N-mm/rad)
- $\beta_{sec}$  = web *distortional stiffness*, including the effect of web *transverse stiffeners*, if any, kip-in./rad (N-mm/rad)

**User Note:** If  $\beta_{sec} < \beta_T$ , Equation A-6-10 is negative, which indicates that torsional *beam* bracing will not be effective due to inadequate web distortional stiffness.

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

$M_r$  = flexural *required strength* using *LRFD load combinations*, kip-in. (N-mm)

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

$M_r$  = flexural *required strength* using *ASD load combinations*, kip-in. (N-mm)

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.

In Equation A-6-9,  $L_b$  need not be taken less than the maximum unbraced length permitted for the beam based upon the flexural required strength,  $M_r$ .

## 2b. Continuous Bracing

For continuous bracing, Equations A-6-9 and A-6-10 shall be used with the following modifications:

- (1)  $L/n = 1.0$ ;
- (2)  $L_b$  shall be taken equal to the maximum unbraced length permitted for the beam based upon the flexural required strength,  $M_r$ ;
- (3) The web distortional stiffness shall be taken as:

$$\beta_{sec} = \frac{3.3Et_w^3}{12h_o} \quad (\text{A-6-13})$$

## 6.4. BEAM-COLUMN BRACING

For beam-columns, the *required strength* and stiffness for the axial force shall be determined as specified in Section 6.2, and the required strength and stiffness for the flexure shall be determined as specified in Section 6.3. The values so determined shall be combined as follows:

- (a) When relative lateral bracing is used, the required strength shall be taken as the sum of the values determined using Equations A-6-1 and A-6-5, and the required stiffness shall be taken as the sum of the values determined using Equations A-6-2 and A-6-6.
- (b) When nodal lateral bracing is used, the required strength shall be taken as the sum of the values determined using Equations A-6-3 and A-6-7, and the required stiffness shall be taken as the sum of the values determined using Equations A-6-4 and A-6-8.
- (c) When torsional bracing is provided for flexure in combination with relative or nodal bracing for the axial force, the required strength and stiffness shall be combined or distributed in a manner that is consistent with the resistance provided by the element(s) of the actual bracing details.

## APPENDIX 7

### ALTERNATIVE METHODS OF DESIGN FOR STABILITY

This appendix presents alternatives to the *Direct Analysis Method* of design for stability, defined in Chapter C. The two alternative methods covered are the *Effective Length Method* and the *First-Order Analysis Method*.

The appendix is organized as follows:

- 7.1. General Stability Requirements
- 7.2. Effective Length Method
- 7.3. First-Order Analysis Method

#### 7.1. GENERAL STABILITY REQUIREMENTS

The general requirements of Section C1 shall apply. As an alternative to the *Direct Analysis Method* (defined in Sections C1 and C2), it is permissible to design structures for stability in accordance with either the *Effective Length Method*, specified in Section 7.2, or the *First-Order Analysis Method*, specified in Section 7.3, subject to the limitations indicated in those sections.

#### 7.2. EFFECTIVE LENGTH METHOD

##### 1. Limitations

The use of the *Effective Length Method* shall be limited to the following conditions:

- (1) The structure supports gravity loads primarily through nominally-vertical columns, walls, or frames.
- (2) The ratio of maximum second-order drift to maximum first-order drift (both determined for *LRFD load combinations* or 1.6 times *ASD load combinations*) in all stories is equal to or less than 1.5.

##### 2. Required Strengths

The *required strengths* of components shall be determined from analysis conforming to the requirements of Section C2.1, except that the stiffness reduction indicated in C2.1(2) shall not be applied; the nominal stiffnesses of all structural steel components shall be used. Notional loads shall be applied in the analysis in accordance with Section C2.2b.

**User Note:** Since the condition specified in Section C2.2b(4) will be satisfied in all cases where the effective length method is applicable, the notional load need only be applied in gravity-only load cases.

### 3. Available Strengths

The *available strengths* of members and connections shall be calculated in accordance with the provisions of Chapters D, E, F, G, H, I, J and K, as applicable.

The *effective length factor*,  $K$ , of members subject to compression shall be taken as specified in (a) or (b), below, as applicable.

(a) In *braced frame* systems, *shear wall* systems, and other structural systems where lateral stability and resistance to lateral loads does not rely on the flexural stiffness of columns, the *effective length factor*,  $K$ , of members subject to compression shall be taken as **unity**, unless rational analysis indicates that a lower value is appropriate.

(b) In *moment frame* systems and other structural systems in which the flexural stiffnesses of columns are considered to contribute to lateral stability and resistance to lateral loads, the *effective length factor*,  $K$ , or elastic critical buckling stress,  $F_e$ , of those columns whose flexural stiffnesses are considered to contribute to lateral stability and resistance to lateral loads shall be determined from a sidesway buckling analysis of the structure;  $K$  shall be taken as 1.0 for columns whose flexural stiffnesses are not considered to contribute to lateral stability and resistance to lateral loads.

Exception: It is permitted to use  $K=1$  in the design of all columns if the ratio of maximum second-order drift to maximum first-order drift (both determined for *LFRD load combinations* or 1.6 times *ASD load combinations*) in all stories is equal to or less than 1.1.

**User Note:** Methods of calculating the effective length factor  $K$  are discussed in the *Commentary*.

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

**User Note:** Methods of satisfying the bracing requirement are provided in Appendix 6. The requirements of Appendix 6 are not applicable to bracing that is included in the analysis of the overall structure as part of the overall load-resisting system.

## 7.3. FIRST-ORDER ANALYSIS METHOD

### 1. Limitations

The use of the *First-Order Analysis Method* shall be limited to the

following conditions:

- (1) The structure supports gravity loads primarily through nominally-vertical columns, walls, or frames.
- (2) The ratio of maximum second-order drift to maximum first-order drift (both determined for *LRFD load combinations* or 1.6 times *ASD load combinations*) in all stories is equal to or less than 1.5.

**User Note:** The ratio of second-order drift to first-order drift in a story may be taken as the  $B_2$  multiplier, calculated as specified in Appendix 8.

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

$$\alpha P_r \leq 0.5 P_y \quad (\text{A-7-2-1})$$

where

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

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

$P_y$  = member yield strength ( $=F_y A$ ), kips (N)

## 2. Required Strengths

The *required strengths* of components shall be determined from a *first-order analysis*, with additional requirements (1) and (2) below. The analysis shall consider flexural, shear, and axial member deformations, and all other deformations that contribute to displacements of the structure.

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

$$N_i = 2.1\alpha(\Delta/L)Y_{il} \geq 0.0042Y_{il} \quad (\text{A-7-2-2})$$

where

$$\alpha = 1.00 \text{ (LRFD)} \quad \alpha = 1.60 \text{ (ASD)}$$

$Y_{il}$  = gravity load at level  $i$  from the LRFD load combination or ASD load combination, as applicable, kips (N)

$\Delta/L$  = the maximum ratio of  $\Delta$  to  $L$  for all stories in the structure

$\Delta$  = first-order inter-story drift due to the LRFD or ASD load combination, as applicable, in. (mm). Where  $\Delta$  varies over the plan area of the structure,  $\Delta$  shall be the average drift weighted in proportion to vertical load or, alterna-

tively, the maximum drift.  
 $L$  = story height, in. (mm)

The additional lateral load at any level,  $N_i$ , shall be distributed over that level in the same manner as the *gravity load* at the level. The additional lateral loads shall be applied in the direction that provides the greatest destabilizing effect.

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

- (2) The non-sway amplification of beam-column moments shall be considered by applying the  $B_1$  amplifier of Appendix 8 to the total member moments.

**User Note:** Since there is no second-order analysis involved in the *First-Order Analysis Method*, for design by ASD it is not necessary to amplify ASD load combinations by 1.6 before performing the analysis, as required in the *Direct Analysis Method* and the *Effective Length Method*.

### 3. Available Strengths

The *available strengths* of members and connections shall be calculated in accordance with the provisions of Chapters D, E, F, G, H, I, J, and K, as applicable.

The *effective length factor*,  $K$ , of all members shall be taken as unity.

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

**User Note:** Methods of satisfying this requirement are provided in Appendix 6. The requirements of Appendix 6 are not applicable to bracing that is included in the analysis of the overall structure as part of the overall load-resisting system.

## APPENDIX 8 APPROXIMATE SECOND-ORDER ANALYSIS

This appendix provides, as an alternative to a rigorous second-order analysis, a procedure to account for second-order effects in structures by amplifying the required strengths indicated by a first-order analysis.

The appendix is organized as follows:

8.1. Limitations

8.2. Calculation Procedure

### 8.1. LIMITATIONS

The use of this procedure is limited to structures that support gravity loads primarily through nominally vertical columns, walls, or frames, except that it is permissible to use the procedure specified for determining *P-δ effects* for any individual compression member.

### 8.2. CALCULATION PROCEDURE

The required second-order flexural strength,  $M_r$ , and axial strength,  $P_r$ , of all members shall be determined as follows:

$$M_r = B_1 M_{nt} + B_2 M_{lt} \quad (\text{A-8-2-1a})$$

$$P_r = P_{nt} + B_2 P_{lt} \quad (\text{A-8-2-1b})$$

where

$B_1$  = multiplier to account for *P-δ* effects, determined for each member subject to compression and flexure, and each direction of bending of the member in accordance with Section 8.2.1.  $B_1$  shall be taken as 1.0 for members not subject to compression.

$B_2$  = multiplier to account for *P-Δ* effects, determined for each story of the structure and each direction of lateral translation of the story in accordance with Section 8.2.2.

$M_r$  = required second-order flexural strength using LRFD or ASD load combinations, kip-in. (N-mm)

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

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

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

$P_{nt}$  = first-order axial force using LRFD or ASD load combinations,



with the structure restrained against lateral translation, kips (N)  
 $P_{lt}$  = first-order axial force using LRFD or ASD load combinations,  
 due to lateral translation of the structure only, kips (N)

**User Note:** Equations A-8-2-1 are applicable to all members in all structures. Note, however, that  $B_1$  values other than unity apply only to moments in beam columns;  $B_2$  applies to moments and axial forces in components of the *lateral load-resisting system* (including columns, beams, bracing members and shear walls). See Commentary for more on the application of Equations A-8-2-1.

## 1. Multiplier $B_1$ for $P$ - $\delta$ Effects

The  $B_1$  multiplier for each member subject to compression and each direction of bending of the member is calculated as follows:

$$B_1 = \frac{C_m}{1 - \alpha P_r / P_{e1}} \geq 1 \quad (\text{A-8-2-2})$$

where

$$\alpha = 1.00 \text{ (LRFD)} \quad \alpha = 1.60 \text{ (ASD)}$$

$C_m$  = a coefficient assuming no lateral translation of the frame determined as follows:

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

$$C_m = 0.6 - 0.4(M_1 / M_2) \quad (\text{A-8-2-3})$$

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.

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

$P_{e1}$  = elastic critical buckling strength of the member in the plane of bending, calculated based on the assumption of no lateral translation at the member ends, kips (N)

$$P_{e1} = \frac{\pi^2 EI^*}{(K_1 L)^2} \quad (\text{A-8-2-4})$$

where

$EI^*$  = flexural rigidity required to be used in the analysis  
 (=  $0.8\tau_b EI$  when used in the direct analysis method where  $\tau_b$  is as defined in Chapter C; =  $EI$  for the effective length and first-order analysis methods.)  
 $E$  = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)  
 $I$  = moment of inertia in the plane of bending, in.<sup>4</sup> (mm<sup>4</sup>)  
 $L$  = length of member, in. (mm)  
 $K_1$  = effective length factor in the plane of bending, calculated based on the assumption of no lateral translation at the member ends, set equal to 1.0 unless analysis justifies a smaller value

It is permitted to use the first-order estimate of  $P_r$  (i.e.,  $P_r = P_{nt} + P_{lt}$ ) in Equation A-8-2-2.

## 2. Multiplier $B_2$ for $P$ - $\Delta$ Effects

The  $B_2$  multiplier for each story and each direction of lateral translation is calculated as follows:

$$B_2 = \frac{1}{1 - \frac{\alpha P_{story}}{P_{e story}}} \geq 1 \quad (\text{A-8-2-5})$$

where

$$\alpha = 1.00 \text{ (LRFD)} \quad \alpha = 1.60 \text{ (ASD)}$$

$P_{story}$  = total vertical load supported by the story using LRFD or ASD load combinations, as applicable, including loads in columns that are not part of the *lateral load-resisting system*, kips (N)

$P_{e story}$  = elastic critical buckling strength for the story in the direction of translation being considered, kips (N), determined by sidesway buckling analysis or as:

$$P_{e story} = R_M \frac{H L}{\Delta_H} \quad (\text{A-8-2-6})$$

where

$$R_M = 1 - 0.15 (P_{mf}/P_{story}) \quad (\text{A-8-2-7})$$

$L$  = story height, in. (mm)

$P_{mf}$  = total vertical load in columns in the story that are part of *moment frames*, if any, in the direction of translation being considered (= 0 for *braced frame* systems), kips (N)

$\Delta_H$  = first-order inter-story drift, in the direction of translation being considered, due to lateral forces, in. (mm), computed

using the stiffness required to be used in the analysis (stiffness reduced as provided in Section C2.3 when the Direct Analysis Method is used). Where  $\Delta_H$  varies over the plan area of the structure, it shall be the average drift weighted in proportion to vertical load or, alternatively, the maximum drift.

$H$  = story shear, in the direction of translation being considered, produced by the lateral forces used to compute  $\Delta_H$ ,  
kips (N)

**User Note:**  $H$  and  $\Delta_H$  in Equation A-8-2-6 may be based on any lateral loading that provides a representative value of story lateral stiffness  $H/\Delta_H$ .